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Water-Supply and Irrigation Paper No. 89 Series J, Water Storage, 9

DEPARTMENT OF THE INTERIOR
UNITED STATES GEOLOGICAL SURVEY
CHARLES D. WALCOTT, DIRECTOR

WATER RESOURCES

OF THE

SALINAS VALLEY, CALIFORNIA

BY

HOMER HAMLIN

WASHINGTON
GOVERNMENT PRINTING OFFICE
1904

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
HYDROGRAPHIC BRANCH,
Washington, D. C., July 22, 1903.

SIR: I have the honor to transmit herewith, for publication in the Water-Supply and Irrigation series, a report on the water resources of the Salinas Valley, California, by Mr. Homer Hamlin.

The report relates particularly to that portion of the Salinas Valley which lies within the boundaries of Monterey County. It describes the topography, hydrography, and economic geology of a portion of the region and discusses the methods of irrigation and the irrigation systems, the extent of underground waters, and the possibility of extending irrigation by the utilization of surface streams and the impounding of flood waters.

Very respectfully,

F. H. NEWELL,
Chief Engineer

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

WATER RESOURCES OF THE SALINAS VALLEY, CALIFORNIA.

By HOMER HAMLIN.

INTRODUCTION.

This report is based on investigations made by the United States Geological Survey, cooperating with the board of supervisors of Monterey County and the California Water and Forest Association, in Monterey County, Cal., during 1900, 1901, and 1902. The work was done under the supervision of Mr. J. B. Lippincott, resident hydrographer of the United States Geological Survey.

The report relates particularly to that portion of the Salinas Valley that lies within the boundaries of Monterey County, and describes the topography, hydrography, and economic geology of a portion of the region. It also discusses the extent of the underground water, the methods of irrigation, the irrigation systems, and the possibility of extending irrigation by the utilization of surface streams and by impounding flood waters.

The first investigation made in this region was a reconnaissance conducted under the direction of Prof. Charles D. Marx in the summer of 1900. During this reconnaissance, which was made for the purpose of locating and examining the reservoir sites in the drainage basin of Salinas River, Professor Marx's party explored the drainage basins of the Arroyo Seco, of San Antonio and Nacimiento rivers, and a portion of the drainage basin of San Lorenzo Creek. Six reservoir sites were discovered, and preliminary surveys and estimates were made to determine the capacity of each and the cost of storing water. Five of these reservoir sites are in the drainage basin of the Arroyo Seco, viz, the Pettitt, Leigh, Currier, Foster, and The Pools sites. One reservoir site, the Pinkerton, was discovered on the San Antonio, and one, the Mathews, on San Lorenzo Creek

In October, 1900, Mr. W. W. Cockins, jr., was put in charge of the study of underground waters in the vicinity of Salinas, and during the winter of 1900–1901 was also in charge of stream measurements on Salinas River and its tributaries. In December, 1900, he established

a gaging station on San Antonio Creek near Jolon, one on San Lorenzo Creek at the Hollenbeck ranch, and one on Arroyo Seco at Pettitt's ranch. A gaging station had been established on Salinas River prior to 1900.

In March, 1901, the writer was detailed to study the hydrography and geology of a portion of this region. The field work extended over a period of six months.

In the Salinas Valley, as in many parts of California, pumping water for irrigation is expensive on account of the high price of fuel. The object of the geologic investigation was to discover, if possible, cheap fuel in the form of coal or oil. A reconnaissance was made of the region and detailed examinations were made of certain localities. The result was disappointing, as no territory that will yield oil in commercial quantities was found, and the one coal field is of small extent and almost inaccessible.

In presenting the report on the economic geology of the region the writer wishes to disclaim any idea of its being more than a preliminary description of the conditions observed during the reconnaissance. At no place were the investigations exhaustive, and facts relating to the oil-producing rocks, coal beds, and water supply only were noted. Much yet remains to be done in the study of the general geology of the region and the correlation of the various formations with equivalent strata in other regions.

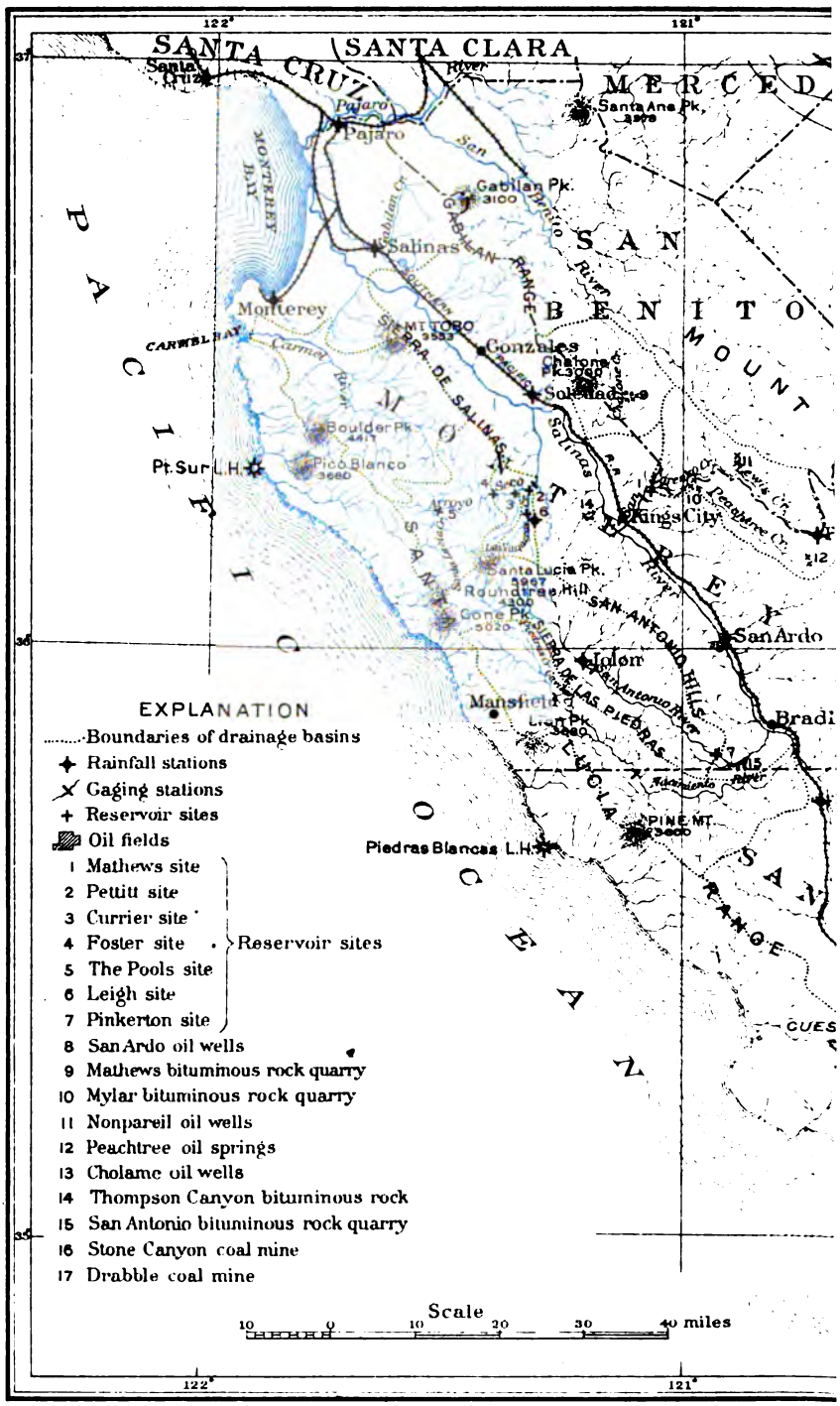
In October, 1901, Mr. H. E. Green was detailed to make topographic surveys of the Mathews reservoir and dam site. He also designed the dam and made estimates of the cost of construction and of the capacity of the reservoir. In August and September, 1902, the writer made a topographic survey of the Currier and Foster reservoir and dam sites, and later designed the dams and made estimates of the cost of construction and of the capacity of the reservoirs.

General acknowledgment is due a large number of persons who have rendered assistance. Thanks are particularly due Dr. William H. Dall for the identification of fossils; to the United States Department of Agriculture for well records and maps of a portion of the Salinas Valley; to the Spreckels Sugar Company for statistics on irrigation; to Mr. Lou G. Hare, county surveyor, for use of county maps, and to Messrs. T. T. Tidball, J. W. Leigh, A. J. Meyers, A. Hubbard, J. M. Krenkel, C. H. Royse, and Mrs. M. Griffin, who have furnished rainfall records.

GEOGRAPHY.

The total area of the Salinas Valley is 4,780 square miles. Its length is 150 miles and its maximum width about 45 miles. (See Pl. I.) It extends from the high mountain region in southeastern San Luis Obispo County to the Bay of Monterey, 80 miles south of San Francisco. The Salinas Valley, in common with the other Coast Range

U.S. GEOLOGICAL SURVEY



EXPLANATION

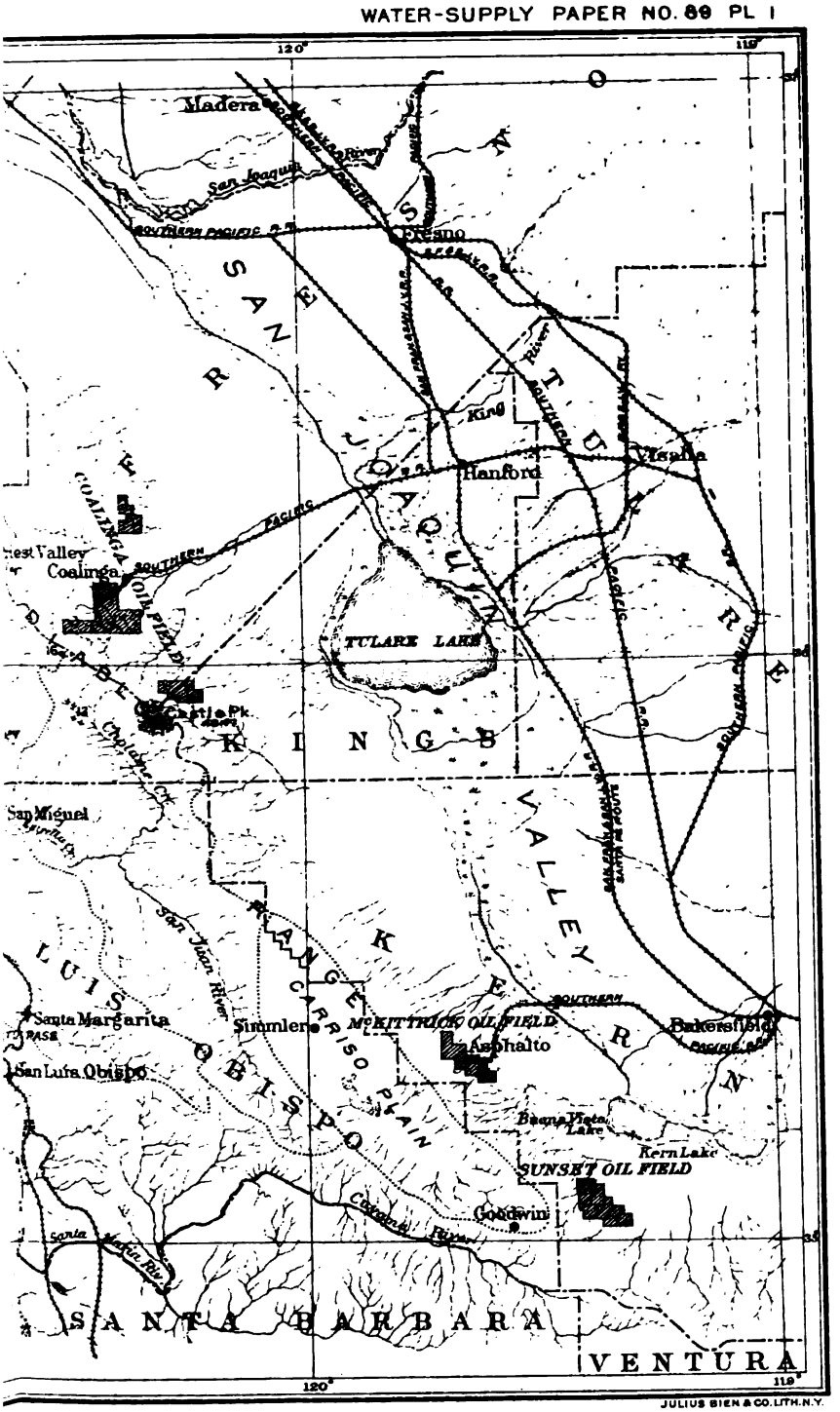
- Boundaries of drainage basins
- Rainfall stations
- Gaging stations
- Reservoir sites
- Oil fields

Reservoir sites:
1. Mathews site
2. Pettitt site
3. Currier site
4. Foster site
5. The Pools site
6. Leigh site
7. Pinkerton site

8. San Ardo oil wells
9. Mathews bituminous rock quarry
10. Mylar bituminous rock quarry
11. Nonpareil oil wells
12. Peachtree oil springs
13. Cholame oil wells
14. Thompson Canyon bituminous rock
15. San Antonio bituminous rock quarry
16. Stone Canyon coal mine
17. Drabble coal mine

Scale
10 0 10 20 30 40 miles

MAP OF THE DRAINAGE B
SHOWING HYDRO



ASIN OF THE SALINAS RIVER
)GRAPHIC FEATURES

3 miles from the crest of the range to the ocean, and the greatest width is but 9 miles. The highest peaks in this range are Santa Lucia, altitude 5,967 feet; Boulder Peak, altitude 4,417 feet; Pico Blanco, altitude 3,680 feet; Roundtree Hill, altitude 4,300 feet, and Pine Mountain, altitude 3,600 feet. The average altitude of the crest can not be less than 3,500 feet, and is probably nearer 4,000 feet.

Between the Salinas Valley and the high Santa Lucia Range are other mountain ranges, the most northern of which is known as the Sierra de Salinas (Pl. I). This is a long mountain ridge extending from the mouth of the canyon of Arroyo Seco, in Monterey County, northwestward about 25 miles to the mesa lands lying southwest of Salinas. It presents steep slopes toward the Salinas Valley, shutting out from view the higher and more rugged ranges nearer the coast. The highest point, Mount Toro, altitude 3,553 feet, is a broad, rounded ridge. This range probably has an average altitude of nearly 2,500 feet throughout its entire length.

The mountain range known as the San Antonio Hills (Pl. I) is a divergent ridge sent off from the most elevated portion of the Santa Lucia Range. It terminates rather suddenly near Bradley, where San Antonio River turns northeastward and joins the Salinas. Although not conspicuous as a topographic feature it can be traced for several miles beyond this point in the low rolling ridge northeast of San Miguel, in San Luis Obispo County. The range has been deeply eroded and many canyons extend far back toward the rounded crest, the average altitude of which is from 2,000 to 2,500 feet.

Between the San Antonio Hills and the main Santa Lucia Range is the range known as the Sierra de las Piedras (see Pl. I). It lies between San Antonio and Nacimiento rivers and, like the San Antonio Hills to the northeast, is a divergent ridge from the main range. It extends southeastward to the point where the Nacimiento turns northeastward to join Salinas River, but can be traced geologically to the San Jose Mountains east of the Santa Margarita Valley, in San Luis Obispo County. Its average altitude is 2,000 to 2,500 feet, and it is an inconspicuous feature in the general topography of the region, lying as it does just northeast of the main mountain ridge. In the past there has been much confusion regarding the names of these different ranges.

The Salinas Valley is the largest of the intermontane valleys of the Coast Range region. Along the coast it is a broad, fertile valley ranging from 6 to 10 miles wide, gradually narrowing to about 5 miles at Soledad, 30 miles inland. For 25 miles of this distance the valley is walled in by the steep slopes of the Sierra de Salinas and the Gabilan Range, but both drop down near Soledad and the valley widens on the northeast side into a broad mesa, or elevated plain, lying between the Santa Lucia and Mount Diablo ranges. This mesa extends far to the southeast and ends in the arid Carriso Plain, lying east of San

Juan River. The mesa has an average width of about 20 miles and rises gently from Salinas River far up the slopes of the Mount Diablo Range. It has been greatly dissected by numerous canyons, which traverse it in a general southwesterly direction. This mesa was traced to the east and northeast of Chalone Peak, where it appears to extend across the ridge between San Benito and Salinas rivers.

GEOLOGY.

GENERAL GEOLOGY.

STRUCTURE.

The upper Salinas Valley is a synclinal fold which extends in a northwest-southeast direction between the Mount Diablo and Santa Lucia ranges. The characteristic structure of this fold may be seen by one crossing the valley in a northeasterly direction from the Sierra de las Piedras to the Mount Diablo Range along a line passing through the town of Bradley. The different formations are seen to dip from both sides toward the axis of the fold, in the valley below. In both of the above-mentioned ranges sedimentary rocks fill the troughs of subordinate folds parallel with the main syncline. In general, in and near the mountains, the formations have been compressed into sharp folds, while in the broad Salinas Valley they are practically undisturbed.

STRATIGRAPHY.

Basement complex.—The Basement complex on which all the unaltered sedimentary rocks of the region rest consists of (1) a series of metamorphic rocks—gneiss, schist, marble, etc.; (2) granitic rocks, chiefly granite, often coarsely porphyritic. The granite has been described by Prof. A. C. Lawson as the Santa Lucia granite.[a]

The rocks of the Basement complex form the axes or cores of the Santa Lucia, Gabilan, and Mount Diablo ranges, and evidently extend to a great depth. The age of the metamorphic series is not definitely known, but it is probably as old as the Carboniferous.[b]

San Luis formation.—The name San Luis formation has been applied by Dr. H. W. Fairbanks[c] to a group of sedimentary and associated rocks in the San Luis quadrangle. The formation consists of conglomerates, sandstones, shales, jasper, and altered eruptives. These rocks outcrop along the southwestern slope of the Mount Diablo Range in a long, narrow belt, extending the entire length of Monterey County. They here probably rest unconformably on the Basement complex. Dr. Fairbanks has also described rocks belonging to this formation in the Santa Lucia Range, extending along the coast

[a] Bull. Dept. Geology, Univ. California, vol. 1, 1893, p. 9.
[b] Twenty-second Ann. Rept. U. S. Geol. Survey, pt. 1, 1901, p. 369.
[c] Geologic Atlas U. S., folio 101, U. S. Geol. Survey, 1903.

from Sur River southeastward to San Luis Obispo County.[a] The San Luis formation is considered to be the equivalent of the Franciscan series and is probably of Jurassic age.

Vaquero sandstone.—In the Salinas Valley the Vaquero sandstone is a well-defined formation. So far as observed in this region it rests unconformably on the Basement complex and on stratified terranes older than the Neocene, being thus in this locality the oldest known member of the Neocene; in other localities Neocene formations are found below the Vaquero sandstone, indicating that it is not the basal member of the Neocene.

The Vaquero formation is a rather coarse, uniformly gray, white or light-yellow quartzose sandstone, with an occasional stratum of granitic pebbles. It is of great thickness along the eastern slope of the Santa Lucia Range, especially in Los Vaqueros Valley; hence the designation proposed by the writer for this series of sandstones. In the Mount Diablo Range it appears at a few points beneath later formations. On the summit of the Mount Diablo Range, around the headwaters of Stone Canyon, there is a series of beds beneath the Monterey shale which may be a portion of the Vaquero or may be older. The Stone Canyon coal beds are in this series. The following fossils have been found in the Vaquero sandstone: *Balanus* sp.?; *Mytilus* sp., probably *mathewsonii* Gabb; *Ostrea tayloriana* Gabb (Young)?; *Ostrea titan* Conrad?; *Pecten magnolia* Conrad; *Turritella hoffmanni* Gabb?; *Chione mathewsonii* Gabb; *Chione* n. sp. (large, characteristic of this horizon); *Mactra* aff. *catilliformis* Conrad; *Pecten estrellanus* Conrad; *Pecten* (*Chlamys*) n. sp., S.; *Pecten* (*Plagioctenium*) n. sp., A.

Monterey shale.—Above the Vaquero sandstone is found the Monterey shale, which, by reason of its peculiar lithologic characteristics, is conspicuous among the rocks of the Coast Range. In Monterey County it is found in a broad belt along the northeastern slope of the Santa Lucia Range and at frequent intervals along the southwestern slope of the Mount Diablo Range. In both ranges it extends southward far beyond Monterey County. It is usually found resting on the Vaquero sandstone, but is also in contact with older formations at several points.

This formation consists of an enormous thickness of thin-bedded siliceous shales, with a small amount of limestone and cherts, especially near the base. The shale beds vary in thickness from a fraction of an inch up to 2 feet, and in color from white, through brown and chocolate color, to black. The more porous shales tend to weather white, while the cherts remain brown or black, and the limestone weathers a rusty brown.

When first exposed these shales and limestones are dark, often black, with disseminated bituminous and carbonaceous matter. If closely examined under a microscope or with a magnifying glass, they

[a] Jour. Geol., vol. 3, 1895, pp. 416-425.

are seen to contain vast numbers of minute fossil shells and fish scales. These fossils are the remains of marine animals, and show that the shales were deposited in the ocean. It appears probable that the organic remains buried in the Monterey shale are in part, at least, the primary source of the oil and bitumen found in Neocene rocks in California.

At the base of the Mount Diablo Range, in Cholame Valley, is found a thick series of sandstones and shales, resting on the Monterey shale. Conspicuous in this series are beds of bluish-gray sandstone and shale much resembling the Monterey shale below. There are many alternations of the sandstone and shale beds, both of which contain certain well-marked fossil forms. In Priest Valley there is a thick series of sandstone and shale very similar in appearance to the series in Cholame Valley, but it contains, in addition to the sandstone and shale beds noted above, two thin beds of impure lignite. Some of the fossils in the Priest Valley region are identical with those in Cholame Valley. The field work was not done in enough detail to determine the stratigraphic relations of these beds. Our present knowledge only warrants us in saying that both belong to the Neocene and are probably a portion of what is known as the Miocene.

Santa Margarita formation.—Above the Monterey shale on the southwest side of the Salinas Valley is found a series of rocks to which Mr. H. W. Fairbanks[a] has given the name Santa Margarita formation. It has a most characteristic fauna, and is probably the equivalent of the San Pablo formation, described by Dr. J. C. Merriam.[b] In the Santa Lucia Range, where it outcrops at frequent intervals northeast of the Monterey shale belt, it consists of soft gray sandstone. On the northeast side of the Salinas Valley the formation is sandstone at the base, grading upward into soft chalk-like shale and clay shale. It outcrops over a large area in the Salinas Valley near Kings City, where the most characteristic fossil is the large barnacle *Tamiosomia gregaria* Conr. The sandstone beds of this formation, in many localities in California, are saturated with bitumen and asphaltum. The occurrence of such beds in Monterey County is described on page 18. From the incomplete data at hand it seems probable that the Santa Margarita formation is more recent than the sandstones and shales noted in Cholame Valley and in Priest Valley.

Paso Robles formation.—In the upper Salinas Valley is found a series of rocks known as the Paso Robles formation, which has been described by Mr. H. W. Fairbanks.[c] The basal beds of this formation are conglomerates, above which lie alternating beds of reddish clay and conglomerates composed of pebbles of shale, jasper, and granitic rocks. The conglomerates seem to be of local origin and

[a] Geologic Atlas U. S., folio 101, U. S. Geol. Survey, 1903.
[b] Bull. Dept. Geology, Univ. California, vol. 2, 1898, pp. 113-116.
[c] Jour. Geol., vol. 6, 1898, p. 565.

contain a large percentage of Monterey shale pebbles. In the Salinas Valley the rocks of this formation are practically undisturbed, but they are sharply folded in both the Mount Diablo and Santa Lucia ranges. The marked unconformity in bedding between the Paso Robles and all the formations below serves as one of the most satisfactory datum planes in the stratigraphy of the Coast Range. The formation was traced from the upper Salinas into Monterey County, where it covers a large area in the vicinity of Bradley. It is greatly developed in the mesa region on the east side of Salinas River and is found between Monterey and Toro Creek, southwest of Salinas.

Later sandstones.—East of Monterey Bay is found a soft, quartzose sandstone of considerable thickness, yellow or reddish brown in color, and on weathered surfaces often containing small nodules of sand cemented by iron oxide. It disintegrates into the soft sand beds which form the sand hills north and west of Salinas. It appears to have once extended up the Salinas Valley as far as San Lucas, probably as a stream deposit, but most of it has been removed by erosion. This formation is more recent than those that have been described, and rests unconformably on the older rocks.

Stream deposits.—The stream deposits in the lower Salinas Valley consist of unconsolidated beds of clay, sand, gravel, and shale pebbles. In the vicinity of Salinas these river deposits yield large quantities of ground water. The wells penetrate alluvial sands and clays, reaching the water supply in beds of gravel 150 to 200 feet below the surface. A further description of these underground waters is given on page 21.

ECONOMIC GEOLOGY.

OIL AND BITUMINOUS ROCK DEPOSITS.

In Monterey County bituminous sandstone and shale are found at several points. Near San Ardo (Pl. I) the Monterey shale has a total thickness of about 2,200 feet. Both the Monterey shale and the Vaquero sandstone below have been sharply flexed into a secondary anticlinal fold on the northeast limb of the San Antonio Hills anticline. The shale is here very different from the typical Monterey shale in the Santa Lucia Range. Some beds are soft and chalk-like; others contain an unusual percentage of fine sand and mica flakes, interstratified with beds of quartzose sandstone. The whole series is bituminous, but the sand beds are much richer in bitumen than the others. The soft shales weather white, but when broken are seen to be colored black or brown with bitumen. The upper 770 feet of this formation consists of alternating beds of chalk-like shale and sandstone. Between these beds and the basal sandstones of the Santa Margarita formation is from 3 to 5 feet of cherty shale, characteristic of the Monterey. Small tar springs are found in this series at several points, and also in the typical Monterey shale to the southwest.

Above this series to the northeast are found beds of sandstone with shale inclusions and fossils characteristic of the Santa Margarita formation.

The amount of liquid bitumen at this locality is exceedingly small. Both the shales and sandstones are close-textured rocks and would yield oil very slowly even if it should be present in large quantities. The strata are steeply inclined, dipping northeastward at an angle of from 65° to 90°. Oil wells drilled here simply pass down into one or two beds of sand instead of piercing several, the chance of reaching any productive oil sand being proportionately reduced. The rocks bituminized are also without an impervious cover, the crest of the anticline having been removed by erosion.

Several prospect wells have been drilled here, but all have thus far proved unproductive. The Tomboy Oil and Improvement Company drilled one well in sec. 19, T. 22 S., R. 10 E., Mount Diablo meridian. The San Antonio Oil Company drilled on their property in secs. 20, 29, and 30, T. 22 S., R. 10 E., M. D. M., about 2 miles west of San Ardo, and the San Antonio Consolidated Oil Company drilled one well on its property in sec. 12, T. 22 S., R. 9 E., and sec. 7, T. 22 S., R. 10 E., M. D. M.

The Mathews bituminous rock quarry is situated in sec. 29, T. 17 S., R. 8 E., M. D. M., in San Benito County, about 7 miles northeast of Metz. (See Pl. I.) The bituminous rock consists of a bed of coarse sand and bowlders, resting directly upon the eroded surface of the granitic axis of the Gabilan Range, and above this are finer sands, the whole having a considerable but variable thickness. Three prospect holes have been opened and a considerable quantity of bituminous sand rock, which seems to be of excellent quality, has been hauled away. The bituminous sand is covered by thick beds of soft, chalky shale, probably a part of the Monterey shale, resembling that at San Ardo. The shale has a low dip, 3° to 5°, toward the east.

The Mylar bituminous rock quarry is situated in secs. 14 and 15, T. 19 S., R. 9 E., M. D. M., 10 miles northeast of Kings City. The quarry is owned by the City Street Improvement Company, of San Francisco. The bituminous rock is here of considerable thickness and consists of coarse granitic gravels which rest directly upon the worn surface of the granitic axis of the Gabilan Range, followed upward by beds of coarse granitic sand, the whole evidently derived from the wear of the adjacent granite rocks. The gravels and sandstones are bituminous throughout their entire thickness. Resting on the bituminous sandstone are beds of soft chalky shale, probably a portion of the Monterey shale series. The sandstone dips westerly at a low angle, 3° to 5°. Considerable rock has been quarried here and shipped to various localities.

No oil wells have been drilled either at this point or at the Mathews quarry, and it is not probable that wells would be successful, as the

bituminous rock which rests on the eroded surface of the granite is of uncertain thickness and small extent at either locality. There are also beds of similar sandstone, not bituminous, however, at other points along the buried granitic axis of the Gabilan Range. The exact relation of these sands to the shales above is not clearly understood, and they may be beds older than the Monterey shale, which have been impregnated with bitumen by downward infiltration.

About 6 miles northeast of the Mylar quarry the Nonpareil Consolidated Oil Company has drilled several prospect wells in sec. 32, T. 18 S., R. 10 E., M. D. M. These wells were drilled in the Monterey shale east of a great fault which has brought that formation up into contact with the Paso Robles formation. The shales here have a dip of 50° to the northeast and are succeeded above by beds of sandstone. There is a small amount of brea and bituminous sand in the Paso Robles formation along the line of the fault. These wells were not successful and have been abandoned. At several points between this locality and the Peachtree ranch there are indications of oil, but none to justify exploitation. Near the eastern corner of the Peach Tree ranch there is another outcrop of bituminous sandstone and conglomerate, apparently a part of the Paso Robles formation. These bituminous beds rest on crumpled strata of the Monterey shales which are also impregnated with bitumen. The deposit is of no value as bituminous rock, being dry and friable, and there is no probability that a productive oil sand will be reached at this place.

In the East Fork of Little Cholame Creek, northwest of Parkfield (Pl. I), there are many seepages of heavy brea or maltha. These are from the Monterey shale, from the sandstone directly above, and at one point from the Paso Robles formation. The seepages are from thin beds of bituminous sandstone and sandy shale. All of the formations examined here, except the Paso Robles, are close-textured rocks; no open porous sandstones were seen. The strata are greatly disturbed and are in close proximity to the granitic rocks below. The amount of bitumen is small and the conditions are not favorable for its storage.

Several oil wells have been drilled in this region, none of which was successful. The Cholame Valley Oil and Development Company drilled in sec. 31, T. 22 S., R. 14 E.; the Parkfield Oil Company in sec. 16, T. 23 S., R. 14 E., and the Waverly Oil Company in sec. 32, T. 22 S., R. 14 E., M. D. M.

At several points in Monterey and San Luis Obispo counties the basal sandstones of the Santa Margarita formation are bituminous. This sandstone is impregnated with bitumen at the mouth of Thompson Canyon, 4 miles west of Kings City (Pl. I), where it outcrops for some distance in a northwest-southeast direction along the northeast side of the Monterey shale area. Both the sandstone and shale are tilted at a high angle, dipping about 50° NE. This sandstone is now

Table of analyses of coal from Drabble and Stone Canyon mines.

Locality.	Coking qualities.	Water.	Volatile matter.	Fixed carbon.	Ash.	Authority.
Drabble mine:						
Upper tunnel—						
Upper vein	None	11.25	38.25	38.50	12.0	Mathyas.[a]
Lower vein	...do	13.25	37.65	36.60	12.5	Do.[a]
Lower tunnel—						
Upper vein	...do	13.25	33.90	38.60	14.25	Do.[a]
Lower vein	...do	13.00	37.25	31.50	13.25	Do.[a]
Stone Canyon mine:						
From dump	Slight	5.25	39.75	49:25	5.75	Do.[a]
Heading tunnel No. 1	...do	4.15	50.55	41.04	4.26	Hillebrand.[b]

[a] Twelfth Ann. Rept. California State Mining Bureau, 1894, p. 64.
[b] U. S. Geological Survey.

UNDERGROUND WATER.

In October, 1901, Mr. W. W. Cockins, jr., was detailed to study the ground water of the lower Salinas Valley. Data were collected regarding the depth of wells, amount of water, cost, etc. From his observations, supplemented by the work of the Bureau of Soils of the United States Department of Agriculture, the table on pages 22–30 has been compiled.

The location of the wells is shown on Pl. II. In a small bow-shaped area west of Salinas the wells formerly flowed slightly; the boundaries of this artesian tract are indicated on the map. Some of these wells flow at present when not drawn down by pumping. The artesian water is struck in a bed of coarse gravel that lies about 150 feet below the surface. In passing northeastward up the valley this stratum is found at a greater depth and the artesian water rises, but fails to reach the surface. The distance from the surface to the water from the artesian stratum is shown on the map by heavy lines, the line numbered 10 indicating that the ground water will rise to within 10 feet of the surface, but in order to reach the gravel bed it will be necessary to sink 150 to 160 feet plus 10 feet, or 160 to 170 feet, etc. There are not enough deep wells to trace the entire outline of this basin, but the direction taken by the various curves of depth indicate that the gravel bed may thin out in passing up the valley and that they probably extend northwestward beyond Castroville to the Bay of Monterey.

COAL.

There are two localities in Monterey County where coal beds have been exploited, Stone Canyon and Priest Valley. (See Pl. I.) At the Stone Canyon mine (formerly known as Slack Canyon coal mine) the thickness of the coal bed is from 12 to 14 feet. The entire thickness is clean coal without shale partings. This coal bed has been traced, by means of tunnels, pits, and borings, for 4,300 feet along the strike, and to a depth of 526 feet at the incline of the Stone Canyon mine.[a] The analyses made (see table on p. 21) indicate that the coal is a lignite and that it contains a large percentage of volatile matter. The roof of this mine is a hard, yellowish sandstone, which outcrops above the coal bed in massive strata several hundred feet thick. The floor is of soft, friable clay shale, much crushed and full of slickensides. It is of considerable thickness, and west of the mine appears to rest directly upon the San Luis formation, but to the east it rests on sandstone similar to that above the coal bed. The whole series of sandstone, shale, and coal is tilted up at high angles, with northward dips ranging from 80° to 90° along the outcrop. A very large amount of sulphureted hydrogen escapes from the coal bed below the water plane, making it difficult for men to work in the incline.[b]

This mine is in an isolated region on the summit of Mount Diablo Range, 2,600 feet above sea level, and 20 miles from a railroad. The construction of a branch railroad will be expensive and difficult, although a line could be built to within 3 or 4 miles of the mine at a reasonable cost, and the coal delivered to this point by a cable incline operated by gravity.

In Priest Valley and on Wartham Creek there are coal beds which have been prospected at various times (see Pl. I), especially those at the Drabble mine, on Wartham Creek.[c] These mines are now idle, and it is evident from an examination of the mine and region that the coal is of poor quality and too far from market to be of value at present (see table of analyses on next page). The coal beds exposed in the Drabble mine extend northwestward under Priest Valley, where they are folded into a synclinal trough. The same series of beds appears on the southwest side of the valley, dipping northeastward, and also on the northeast side of the valley, dipping southwestward. The coal here is not so good as that at the Drabble mine.

Coal beds are also reported to occur at intervals northwest of Priest Valley, in the Mount Diablo Range,[d] but from reports none of these deposits seemed to be of any economic value and they were not visited.

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[a] Information given by F. J. Horswell, Superintendent.
[b] Eighth Ann. Rept. California State Mining Bureau, 1888, p. 404.
[c] Twelfth Ann. Rept. California State Mining Bureau, 1894, p. 51.
[d] Ibid., p. 61.

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3 miles from the crest of the range to the ocean, and the greatest width is but 9 miles. The highest peaks in this range are Santa Lucia, altitude 5,967 feet; Boulder Peak, altitude 4,417 feet; Pico Blanco, altitude 3,680 feet; Roundtree Hill, altitude 4,300 feet, and Pine Mountain, altitude 3,600 feet. The average altitude of the crest can not be less than 3,500 feet, and is probably nearer 4,000 feet.

Between the Salinas Valley and the high Santa Lucia Range are other mountain ranges, the most northern of which is known as the Sierra de Salinas (Pl. I). This is a long mountain ridge extending from the mouth of the canyon of Arroyo Seco, in Monterey County, northwestward about 25 miles to the mesa lands lying southwest of Salinas. It presents steep slopes toward the Salinas Valley, shutting out from view the higher and more rugged ranges nearer the coast. The highest point, Mount Toro, altitude 3,553 feet, is a broad, rounded ridge. This range probably has an average altitude of nearly 2,500 feet throughout its entire length.

The mountain range known as the San Antonio Hills (Pl. I) is a divergent ridge sent off from the most elevated portion of the Santa Lucia Range. It terminates rather suddenly near Bradley, where San Antonio River turns northeastward and joins the Salinas. Although not conspicuous as a topographic feature it can be traced for several miles beyond this point in the low rolling ridge northeast of San Miguel, in San Luis Obispo County. The range has been deeply eroded and many canyons extend far back toward the rounded crest, the average altitude of which is from 2,000 to 2,500 feet.

Between the San Antonio Hills and the main Santa Lucia Range is the range known as the Sierra de las Piedras (see Pl. I). It lies between San Antonio and Nacimiento rivers and, like the San Antonio Hills to the northeast, is a divergent ridge from the main range. It extends southeastward to the point where the Nacimiento turns northeastward to join Salinas River, but can be traced geologically to the San Jose Mountains east of the Santa Margarita Valley, in San Luis Obispo County. Its average altitude is 2,000 to 2,500 feet, and it is an inconspicuous feature in the general topography of the region, lying as it does just northeast of the main mountain ridge. In the past there has been much confusion regarding the names of these different ranges.

The Salinas Valley is the largest of the intermontane valleys of the Coast Range region. Along the coast it is a broad, fertile valley ranging from 6 to 10 miles wide, gradually narrowing to about 5 miles at Soledad, 30 miles inland. For 25 miles of this distance the valley is walled in by the steep slopes of the Sierra de Salinas and the Gabilan Range, but both drop down near Soledad and the valley widens on the northeast side into a broad mesa, or elevated plain, lying between the Santa Lucia and Mount Diablo ranges. This mesa extends far to the southeast and ends in the arid Carriso Plain, lying east of San

from Sur River southeastward to San Luis Obispo County.[a] The San Luis formation is considered to be the equivalent of the Franciscan series and is probably of Jurassic age.

Vaquero sandstone.—In the Salinas Valley the Vaquero sandstone is a well-defined formation. So far as observed in this region it rests unconformably on the Basement complex and on stratified terranes older than the Neocene, being thus in this locality the oldest known member of the Neocene; in other localities Neocene formations are found below the Vaquero sandstone, indicating that it is not the basal member of the Neocene.

The Vaquero formation is a rather coarse, uniformly gray, white or light-yellow quartzose sandstone, with an occasional stratum of granitic pebbles. It is of great thickness along the eastern slope of the Santa Lucia Range, especially in Los Vaqueros Valley; hence the designation proposed by the writer for this series of sandstones. In the Mount Diablo Range it appears at a few points beneath later formations. On the summit of the Mount Diablo Range, around the headwaters of Stone Canyon, there is a series of beds beneath the Monterey shale which may be a portion of the Vaquero or may be older. The Stone Canyon coal beds are in this series. The following fossils have been found in the Vaquero sandstone: *Balanus* sp.?; *Mytilus* sp., probably *mathewsonii* Gabb; *Ostrea tayloriana* Gabb (Young)?; *Ostrea titan* Conrad?; *Pecten magnolia* Conrad; *Turritella hoffmanni* Gabb?; *Chione mathewsonii* Gabb; *Chione* n. sp. (large, characteristic of this horizon); *Mactra* aff. *catilliformis* Conrad; *Pecten estrellanus* Conrad; *Pecten* (*Chlamys*) n. sp., S.; *Pecten* (*Plagioctenium*) n. sp., A.

Monterey shale.—Above the Vaquero sandstone is found the Monterey shale, which, by reason of its peculiar lithologic characteristics, is conspicuous among the rocks of the Coast Range. In Monterey County it is found in a broad belt along the northeastern slope of the Santa Lucia Range and at frequent intervals along the southwestern slope of the Mount Diablo Range. In both ranges it extends southward far beyond Monterey County. It is usually found resting on the Vaquero sandstone, but is also in contact with older formations at several points.

This formation consists of an enormous thickness of thin-bedded siliceous shales, with a small amount of limestone and cherts, especially near the base. The shale beds vary in thickness from a fraction of an inch up to 2 feet, and in color from white, through brown and chocolate color, to black. The more porous shales tend to weather white, while the cherts remain brown or black, and the limestone weathers a rusty brown.

When first exposed these shales and limestones are dark, often black, with disseminated bituminous and carbonaceous matter. If closely examined under a microscope or with a magnifying glass, they

[a] Jour. Geol., vol. 3, 1895, pp. 416–425.

are seen to contain vast numbers of minute fossil shells and fish scales. These fossils are the remains of marine animals, and show that the shales were deposited in the ocean. It appears probable that the organic remains buried in the Monterey shale are in part, at least, the primary source of the oil and bitumen found in Neocene rocks in California.

At the base of the Mount Diablo Range, in Cholame Valley, is found a thick series of sandstones and shales, resting on the Monterey shale. Conspicuous in this series are beds of bluish-gray sandstone and shale much resembling the Monterey shale below. There are many alternations of the sandstone and shale beds, both of which contain certain well-marked fossil forms. In Priest Valley there is a thick series of sandstone and shale very similar in appearance to the series in Cholame Valley, but it contains, in addition to the sandstone and shale beds noted above, two thin beds of impure lignite. Some of the fossils in the Priest Valley region are identical with those in Cholame Valley. The field work was not done in enough detail to determine the stratigraphic relations of these beds. Our present knowledge only warrants us in saying that both belong to the Neocene and are probably a portion of what is known as the Miocene.

Santa Margarita formation.—Above the Monterey shale on the southwest side of the Salinas Valley is found a series of rocks to which Mr. H. W. Fairbanks[a] has given the name Santa Margarita formation. It has a most characteristic fauna, and is probably the equivalent of the San Pablo formation, described by Dr. J. C. Merriam.[b] In the Santa Lucia Range, where it outcrops at frequent intervals northeast of the Monterey shale belt, it consists of soft gray sandstone. On the northeast side of the Salinas Valley the formation is sandstone at the base, grading upward into soft chalk-like shale and clay shale. It outcrops over a large area in the Salinas Valley near Kings City, where the most characteristic fossil is the large barnacle *Tamiosomia gregaria* Conr. The sandstone beds of this formation, in many localities in California, are saturated with bitumen and asphaltum. The occurrence of such beds in Monterey County is described on page 18. From the incomplete data at hand it seems probable that the Santa Margarita formation is more recent than the sandstones and shales noted in Cholame Valley and in Priest Valley.

Paso Robles formation.—In the upper Salinas Valley is found a series of rocks known as the Paso Robles formation, which has been described by Mr. H. W. Fairbanks.[c] The basal beds of this formation are conglomerates, above which lie alternating beds of reddish clay and conglomerates composed of pebbles of shale, jasper, and granitic rocks. The conglomerates seem to be of local origin and

[a] Geologic Atlas U. S., folio 101, U. S. Geol. Survey, 1903.
[b] Bull. Dept. Geology, Univ. California, vol. 2, 1898, pp. 113-116.
[c] Jour. Geol., vol. 6, 1898, p. 565.

Wells in Salinas Valley—Continued.

No. of well.	Owner of well.	Location.	Class of well.	Depth of well.	Strata.	Depth to water.	Method of lift.	Cost of well.	Cost of machinery.	Use of water.
				Feet.		*Feet.*				
58	Mr. Dedinni	Nacional Rancho	Bored, 2-inch	86		18	Wind			Domestic.
59	A. Abbott	do	Bored, 7-inch	135		20	do			Do.
60	J. Dill	do	Bored, 2-inch	91	Sand, gravel	22	do			Do.
61	Mrs. Jeffery	do	Bored, 7-inch	125		26	do			Do.
62	J. Grau	do	Bored, 2-inch	180			Hand			Do.
63	Mr. Garside	do	Bored, 4-inch	150		25	do			Do.
64	do	do	Bored, 10-inch	25		10	Wind			Domestic and irrigation.
65	Salinas City W. Co	Salinas	Bored, ten 20-inch	200 165		2	Steam			City supply.
66	Monterey Co	County hospital	Dug, 7 by 7	10		2	Wind and horse			Domestic.
67	do	do	Dug, 5 by 5	10		6	Wind			Irrigation.
68	do	do	do	12		10	do			Do.
69	E. Sherwood	Sausal Rancho	Bored, 7-inch	525		45	do	$2,000		Domestic.
70	do	do	Dug, 4 by 4	8		6				
71	C. P. Cooper	Natividad Rancho	Bored, 7-inch	165		46	Wind			Stock and domestic.
72	E. W. Sheehy	do	Bored, 2-inch	180		50	do			Do.
73	H. Mayn	do	do	180		50	do			Do.
74	Mr. Patton	do	Dug, 4 by 4	26		23	do			Stock.
75	J. W. Patton	Natividad	do	10		6	do			
76	G. Mosca	do	Dug, 3 by 3	40		36	Hand			
77	J. Breschini	Blanco	Bored, three 10-inch	135 164		8 7	Wind			Domestic and irrigation.
78	Mrs. C. Clausen	do	Bored, 7-inch	152		7	do	200	$1,700	Domestic.
79	F. McFadden	El Tucho Rancho	Bored, 2-inch	163		7	do			Do.
80	do	do	Bored, two 10-inch	187 198		6.5	Steam	400	2,000	Irrigation.
81	Spreckels Sugar Co	Llano de Buena Vista	Bored, 5-inch	177		20	Hand			Domestic.
82	do	do	Bored, 6-inch			20	Wind			Do.

83	M. Harkens	do	do	100	Clay	30	do		Do.
84	M. T. Martin	do	Bored, 2-inch	142		20	do		Stock and irrigation.
85	J. Grau	do	do	142		17	do		Domestic.
86	do	do	Bored, 10-inch	165		15.5	do	170 80	Irrigation.
87	J. Davila	do		154		16	do		Domestic.
88	Mr. Tavernetti	do	Bored, 10-inch	150		15	Wind and gas engine.		Domestic and irrigation.
89	M. Winn	El Tucho Rancho	Bored, 2-inch	149		7	Wind		Domestic.
90	T. Campbell	do	Bored, 10-inch	178		6	Gas engine		Irrigation.
91	F. Vierra	T. 14 S., R. 2 E., sec. 22	Bored, 2-inch	165		7	Wind		Domestic.
92	Dr. Gonzales	Punta del Monte	Bored, two 10-inch	177		29			
93	Southern Pacific R. R. Co.	Soledad	Dug, 4 by 4	82		31	Wind		Boilers.
94	O. Olsen	T. 18 S., R. 6 E., sec. 29	Dug	280	Sand, clay, and gravel				
95	Willey Bros	T. 18 S., R. 6 E., sec. 21	Dug, 4 by 4	141		137	Wind		Domestic.
96	M. Williams	El Alisal Rancho		90		68	do		Do.
97	J. Duncan	do	Bored, 7-inch	106		36	do	180	Do.
98	P. Zabala	El Alisal Rancho	do	200	Gravel, clay, quicksand, sand.	80	do		Do.
99	D. McKinnon	Sausal Rancho	Bored, 10-inch	240	Sand and clay	50	do		Do.
100		Santa Rita Rancho	Bored	152	Gravel	20			
101		do	Dug	22	do	16			
102		do	Bored	71	do	28			
103		Escarpines Rancho	Dug	80	do	30			
104		do	Bored	160	do	40			
105		do	Dug	70	do	22			
106		do	Bored, 10-inch	105	do	25			
107		Natividad	Dug	42	do	8			
108		Sausal Rancho	Dug, 4 by 4	51		47	Wind		Domestic and stock.
109		Salinas	Bored	180	Gravel	18			
110		do	do	138	do	8			
111		T. 14 S., R. 2 E., sec. 27	do	150	do	12			
112		Morocojo Rancho	do	20	do	15			
113		do	do	205	do	5			
114		do	do	125	do	31			

Wells in Salinas Valley—Continued.

No. of well.	Owner of well.	Location.	Class of well.	Depth of well.	Strata.	Depth to water.	Method of lift.	Cost of well.	Cost of machinery.	Use of water.
				Feet.		*Feet.*				
115		Morocojo Rancho	Bored	180		0				
116		do	Dug	12		6				
117		Castroville	do	18		18				
118		do	do	18		15				
119		do	Bored	80	Gravel	12				
120		do	Bored, 10-inch	107		8				
121		Morocojo Rancho	Dug	15		10				
122		do	Bored	127		12				
123		Coopers	do	112	Gravel	2				
124		San Jon Rancho	do	100	do	6				
125		Salinas	do	101	do	11				
126		do	do	130	do	12				
127		Nacional Rancho	do	90		7				
128		Sausal Rancho	do	100		8				
129		do	do	140		9				
130		San Jon Rancho	do	187	Gravel	8				
131		Cocks Tract	do	180	do	10				
132		do	do	184	do	2				
133		Salinas	do	135	do	15				
134		Nacional Rancho	do	158	do	9				
135		do	do	190	do	9				
136		Salinas	do	105	do	14				
137		do	do	107	do	14				
138		do	do	130	do	11				
139		Nacional Rancho	do	134	do	20				
140		do	Bored, 10-inch	160	do	11				
141		do	Bored	134		12				
142		do	do	134	Gravel	12				

143		do		120	Gravel	12		
144		do		140	do	8		
145		do		140	do	11		
146		Dug	Sensal Rancho	50		20		
147		Bored	Nacional Rancho	90	Gravel	9		
148		do		127	do	11	Irrigation.	
149		do		150		10		
150		do		90	Gravel	7		
151		do		165	do	7		
152		do		160	do	5		
153		do		160	do	8		
154		do		150	do	2		
155		do		140	do	1		
156		do		120	do	2		
157		do		158	do	0.5		
158		do		168	do	8		
159		do		148	do	9		
160		do		162	do	8		
161		do		150	do	15		
162		do		150	do	12		
163		do		95	do	15		
164		do	Riverside	158	do	8		
165		do		164	do	18		
166		do	Llano de Buena Vista	150	do	18		
167		do		151	do	13		
168		do		141	do	13		
169		do		137	do	8		
170		do		128	do	9		
171		do		130	do	15		
172		do		126	do	10		
173		do		170	do	10		
174		do		140	do	25		
175		do		165	do	13		
176		do		130	do	18		
177		do		300	do	14		

Wells in Salinas Valley—Continued.

No. of well.	Owner of well.	Location.	Class of well.	Depth of well.	Strata.	Depth to water.	Method of lift.	Cost of well.	Cost of machinery.	Use of water.
				Feet.		*Feet.*				
178		Llano de Buena Vista	Bored	110	Gravel	10				
179		do	do	160	do	18				
180		do	do	158	do	20				
181		do	do	140	do	18				
182		do	do	120	Sand	8				
183		do	do	70	Gravel	8				
184	Spreckels Sugar Co.	Ranch No. 1	do	150	do	12				
185		Llano de Buena Vista	do	150	do	20				
186		Buena Esperanza	Dug	26	do	8				
187		El Alisal Rancho	do	20	do	16				
188		Buena Vista Rancho	Bored, 7-inch	178	do	50				
189		do	Bored	130	do	22				
190		do	do	65		10				
191		do	do	180		100				
192		do	do	200	Pebbles	60				
193		do	do	120		8				
194		do	do	138	Gravel	0				
195		do	do	100		12				
196		do		106	Gravel, sand	8				
197		El Alisal Rancho	Dug	23		20				
198		do	do	22		18				
199		Buena Vista Rancho	Bored	130	Gravel	10				
200		do	do	136	do	6				
201		Guadalupe Rancho	do	90	do	30				
202		do	do	100	do	8				
203		Nacional Rancho	Bored, 10-inch	180	do	10				
204		Salinas	Bored	778	Clay, sand, gravel, bowlders.					
205		Sec. 30, T. 18 S., R. 6 E.	Springs							

206	Guadalupe Rancho	Dug	45	Gravel	40						
207	Chualar Rancho	do	50	do	45						
208	Buena Esperanza	do	95	do	88						
209	do	Bored	95	do	80						
210	do	Dug	25	do	19						
211	do	Bored	125	do	20						
212	do	Dug	30	do	27						
213	do	do	90	do	28						
214	do	do	20	do	15						
215	do	do	47	do	45						
216	do	Bored	187	do	40						
217	do	do	180	do	90						
218	Chualar Rancho	Dug	50	do	40						
219	Buena Esperanza	do	20	do	15						
220	Allisal Rancho	do	10	do	7						
221	Buena Esperanza	do	20		17						
222	do	do	26	Gravel	22						
223	do	do	60	do	57						
224	do	do	72	do	70						
225	Allisal Rancho	Bored	125	Gravel, clay	40						
226	do	do	163		80						
227	Gabilan Rancho	Dug	40		16						
228	Buena Esperanza	Bored	130	Gravel, loam	123						
229	do	do	246		146						
230	do	Dug	62	Gravel, sand	47						
231	do	Bored	265	Gravel	165						
232	Gonzales	Dug	35		31						
233	do	do	30		28						
234	do	Bored	47		26						
235	do	Dug	187	Blue mud	160						
236	do	do	65	Gravel	30						
237	Punta del Monte	do	60	do	32						
238	do	do	100		90						
239	do	do	100		95						
240	do	do	182	Gravel	175						

Wells in Salinas Valley—Continued.

No. of well.	Owner of well.	Location.	Class of well.	Depth of well.	Strata.	Depth to water.	Method of lift.	Cost of well.	Cost of machinery.	Use of water.
				Feet.		*Feet.*				
241		Punta del Monte	Dug	178	Gravel	170				
242		do	do	32	do	28				
243		do	do	28	do	21				
244		do	do	60		30				
245		Sec. 2, T. 16 S., R. 5 E	do	20	Gravel	10				
246		Zanjones Rancho	do	40		10				
247		Pajare de Sanchez	do	120	Gravel	4				
248		do		80		78				
249		Soledad Rancho		66	Gravel	60				
250		do		67	do	60				
251		do	Dug			15				
252		do	Bored		Gravel	45				
253		do	do	131		20				
254		do	Dug			15				
255		Soledad	Bored	30		27				
256		San Vicente Rancho		250		50				
257		do		50		42				
258		do		44		40				
259		do		30	Gravel, bowlders	24				
260		do		25	Gravel	23				
261		Arroyo Seco Rancho				25				
262		do		100	Gravel, clay	40				
263		do				21				
264		T. 19 S., R. 6 E., sec. 13		59	Clay	40				
265		Posa de los Ositos		70	Sand	30				
266		do				30				
267		San Lorenzo Rancho			Gravel, quicksand	16				
268		do				14				
269		do		200		170				
270		do		40		20				

NOTE.—Well records Nos. 1 to 99, inclusive, obtained by W. W. Cockins, jr.; well records Nos. 100 to 270, inclusive, obtained by Bureau of Soils, Agricultural Department.

The character of the strata filling the lower portion of the valley is shown by a well (No. 204 in table) drilled near the court-house at Salinas, at an altitude of about 33 feet above sea level.

Record of Salinas well.

Character of strata.	Thickness.	Depth of bottom of formation.
	Ft. In.	Ft. In.
Alluvium	4 0	4 0
Yellow clay	6 0	10 0
Yellow sand	8 0	18 0
Yellow clay	5 0	23 0
Yellow quicksand	18 0	41 0
Yellow clay	16 0	57 0
Blue sand	5 0	62 0
Blue clay	10 0	72 0
Yellow clay (small flow of gas)	10 0	82 0
Brown vegetable mold (strong flow of gas, but well soon filled with water)	3 0	85 0
Sand and coarse gravel	10 0	95 0
Blue clay	5 0	100 0
Yellow clay	25 0	125 0
Sand and gravel alternating	57 0	182 0
Quicksand	10 0	192 0
Decomposed coarse gravel	10 0	202 0
Tough yellow clay	2 0	204 0
Blue clay	28 0	232 0
Black sand and mud	3 0	235 0
Rotten gravel	2 0	237 0
Bluish sand	10 0	247 0
Blue clay	17 0	264 0
Yellow sand and coarse gravel	43 0	307 0
Tough yellow clay	1 0	308 0
Yellow sand and coarse gravel	50 0	358 0
Tough yellow clay	10 0	368 0
Yellow sand	9 0	377 0
Coarse gravel and sand	95 0	472 0
Tough whitish clay	7 0	479 0
Gray sand and gravel	15 0	494 0
Seam of limestone	0 4	494 4
Tough yellow clay	51 0	545 4
Mud	0 8	546 0
Sand and gravel	156 0	702 0
Yellow clay	5 0	707 0
Sand and coarse gravel	15 0	722 0

Record of Salinas well—Continued.

Character of strata.	Thickness.	Depth of bottom of formation.
	Ft. In.	Ft. In.
Yellow clay	6 0	728 0
Gravel and bowlders	15 0	743 0
Tough clay and limestone	10 0	753 0
Yellow sand	8 0	761 0
Clay, gravel, and cement	4 0	765 0
Cement sand	10 0	775 0
Dry yellow sand	2 0	777 0
Gray clay with limestone	1 0	778 0

The gravel brought to the surface consists of small waterworn pebbles from half an inch to 2 or 3 inches in diameter; many of the coarser pebbles were broken in the well.[a]

It is probable that the abundant supply of ground water is struck in the beds of sand and gravel from 125 to 202 feet in depth. The well appears to have pierced the clays and limestone of the older formation at a depth of 472 feet, or about 440 feet below sea level. This is then an approximate measure of the depth of the detritus filling this portion of the valley.

In 1880 a well was drilled at Castroville to a depth of 178 feet, producing a volume of fresh water, which at high tide flowed in large quantity over the casing and at low tide ceased flowing. This well is near the mouth of Salinas River, and the surface of the ground is 20 feet above the river.[a]

The gathering ground for this body of underground water is unknown, but the close agreement of the basin with the outcrops of the sand hills on both sides of the valley suggests that this formation may be the source from which the water is indirectly derived by seepage. This idea is strengthened by the fact that the water from the deep wells is of about the same purity as that found in the sand hills, being far purer than the river water, the surface water in this portion of the valley, or the water derived from the older formations. This water may possibly be derived by percolation through deeply buried gravel beds, from the Arroyo Seco or the upper Salinas, but deep borings have not proved that conditions are favorable to this origin.

Southeast of Salinas, as far as Kings City, there are a few pumping plants used for irrigation, some of which draw water from the river. They are described on page 80. The wells in this portion of the valley do not furnish sufficient data on which to base an opinion as to

[a] Tenth Ann. Rept. California State Mining Bureau, 1890, p. 346.

the underground conditions other than that the supply of ground water is limited except along the river.

The only artesian well noted in Monterey County, except in the vicinity of Salinas, is on the Pleyto ranch (Pl. I), at Mr. Pinkerton's place, one-half mile southeast of Pleyto. The total depth of this well is 145 feet. At depths between 70 and 100 feet the drill was in hard, black rock, and at 100 feet water rose 9 feet. At 145 feet a bed of hard rock was penetrated and the water then flowed over the top of a 7-inch casing.[a] This well is situated in the lower end of a synclinal basin, at a point where one would expect to secure an artesian flow if there is one in that locality. This basin extends northwestward about 20 miles, and there is a possibility that a small artesian flow may be struck in the southeast part of it, between Jolon and Pleyto. At no other point in Monterey County were conditions favorable for artesian wells observed.

It seems probable to the writer that deep wells in the mesa region on the east side of the valley, southeast of Kings City, may yield sufficient water for stock-raising purposes if pumped. This can be proved only by experiment, however.

CLIMATE.

CLIMATIC FACTORS.

In an appendix to a report on the climatology of the arid regions of the United States with reference to irrigation Mr. A. W. Glassford makes the following statements:[b]

Two influences dominate the climate of California, radically dissimilar in every particular, combining in ever-varying forces to produce the resultant which is recorded by observers of the weather. One is the sea, tending always to charge the air with moisture, the other the mountain mass, tending always to discharge the moisture from the air. The combination of these two activities in varying proportions is responsible for the variation in the amount of precipitation, including months of drought. It is necessary to consider these two active and determining forces not merely in their resultant, but, so far as is possible, by resolution into their component forces as well. In the present state of knowledge the resolution can not be complete, yet the extent to which it can be made affords interesting results.

The mountain factor.—The States of California and Nevada abut upon the maximum extension in latitude of the Cordilleran system, by which designation is inclusively implied all those ranges, basins, and valleys which in a looser description are often spoken of as the backbone of the continent and considered to include everything from the eastern ranges of the Rocky Mountains to the Pacific Ocean. Between the parallels of 35° and 40° this system attains not only its greatest breadth, but its greatest general elevation. It extends from eastern Colorado across four States and into the ocean, where but a few miles from the California coast it breaks short off from the continental shelf and plunges to abysmal depths.

[a] Information furnished by W. Pinkerton.

[b] Irrigation and water storage in the arid region: House Ex. Doc. 287, Fifty-first Congress, second session, pp. 334-337.

Not only is its width greatest between these parallels, and therefore productive of its maximum influence upon the general circulation of the atmosphere, but also by the massing of many of its extreme heights within these same limits it exerts such violent influence of perturbation as is due to sudden uplifting of air bodies to great altitudes. Thus in Colorado there is a chain of peaks all rising to a height of more than 14,000 feet, of which Pikes Peak is the eastern outpost; Utah and Nevada form the Great Basin, on a general level of 5,000 feet; in California the Sierra Nevada has its peaks of 14,000 feet, as well as Colorado, and at the very edge of the sea is another range of mountains lower than the Sierras, yet of marked influence upon the climate and the rainfall in particular. These systems within the limits of the two States now under examination may properly claim more detailed investigation.

The characteristic orographic feature of this region is the Sierra Nevada, and it is as well the predominant climatic instrument both for California, to which it gives the rain, and for Nevada, from which it withholds it. The geographer and the geologist unite in considering this the most interesting and important link in the Cordilleran system, and the climatologist must unhesitatingly and without reserve give adhesion to their judgment. In brief description it is a long and elevated mountain chain, on the whole the most conspicuous on the continent. It displays its greatest prominence when viewed from the west, because on this side it falls almost to sea level, while upon its eastern slope it merges into the general high altitudes of the interior plateau. But it does not, however, border immediately on the ocean, since for all its course there lies between its foothills and the sea beach a chain of lower mountains, known as the Coast Ranges. * * *

These, the Coast Ranges, are sufficiently important in their relations to precipitation to merit more close consideration. The most important part of the Coast Ranges is that which fences off the great valley of the Sacramento and San Joaquin rivers from the sea. In this portion of their length, where they may be clearly distinguished from all series of inosculating elevation, they have a length of fully 400 miles, and in width vary from 40 to 70 miles; in this particular it is to be noted that their eastern limit is fixed with considerable precision of definition at a practically constant distance from the western limit of the Sierras; the expansions are uniformly made by encroachments upon the sea. The system comprises a multitude of subordinate ranges, some large, some small, but almost all distinguished by names bestowed upon them during the former Spanish occupation of the country, with a few Indian names yet preserved for characteristic peaks. The general trend of the subranges, as of the system at large, is with a tendency toward parallelism with the coast. In proportion to distance from San Francisco, where the system is broken through at sea level by a gap but a mile in width, the summits and the general elevation are found to be higher, and this is true both north and south. * * * From Mount Hamilton the ridge of 3,000 feet elevation continues without interruption and almost in a right line to the Tehachapi country and its coalescence with the same level of the Sierra system. * * *

Having thus indicated the orographic skeleton of the country, it comes next in order to examine the valleys infolded between these mountains.

As it is the greatest, so is the Great Valley of California the most important; it frequently takes the names of the rivers which traverse it and is known in its northern portion as the Sacramento Valley, and in its southern half as the San Joaquin Valley. It is fenced on the east by the Sierra Nevada, on the west by the Coast Ranges, and at north and south by the coalescence of its side walls. Between these walls it has a length of about 450 miles and maintains the average breadth of 40 miles. * * *

In the Coast Ranges are many fertile valleys, which vary greatly in size and conditions, according to position. * * * North of the [San Francisco] bay the

valleys uniformly open into the Sacramento Valley, and each has a name which has nearly the value of a trade-mark in the markets for farm, orchard, and vineyard produce. * * * South of the bay, on the dry eastern slope of the Coast Ranges, not a valley is to be found of any moment. West of the summits are to be found several fertile valleys. Of these the valley of Santa Clara and Alameda open on the Bay of San Francisco, and the valleys of the San Benito and the Salinas open on the Pacific at the Bay of Monterey. * * *

The oceanic factor.—This presentation of the mountain masses of the region under study has been made for the purpose of showing what influences may be counted on as constantly exerted to discharge the moisture from the atmosphere. Another influence is constantly exerted to charge the atmosphere with moisture, and this influence should be examined in turn. It is found in the Pacific Ocean, which washes the entire coast of California. * * *

The largest of all the oceans, the Pacific, is least subject to perturbing influences of a local character. Its conditions are constant over large areas; its currents, both of wind and water, are drawn in broad, sweeping curves, in which extent of space and time of passage serve to override all mere local or temporary modifications. Thus it is enabled to present almost the ideal problem of oceanic circulation and to array upon the climate of California, and in a modified degree that of Nevada, a few masses of simple influence which become involved and difficult of study only through the continental disturbances.

Without interruption that part of the North Pacific Ocean which may be considered as modifying the climate of California stretches away over nearly 100 degrees of longitude. To the west it is bounded by the extreme Orient. * * * The northern limit is drawn by the Aleutian Islands, and the eastern border is the shore of North America. To the south no consistent mass of land appears to hem this ocean in, yet the barrier is none the less strong because it may be measured only with the instruments of the meteorologist. It exists at the thirtieth parallel of north latitude. Below this bounding line is the region of northeast trade winds and the westward drift of the equatorial current, and these two serve sufficiently to bound in wind and water the great basin above. * * *

The winds upon this basin are of the system of the passage winds which are developed upon the surface of the earth by the descent from high altitudes of upper currents. In general these winds vary with the latitude from southwest, westerly, to northwest. * * *

The Californian parallels lie entirely within the northern zone of the passage or antitrade winds, and are therefore under an atmosphere with a uniformly eastern progression as a part of the general system of atmospheric circulation of the globe. * * *

The wind drawn in from sea by the general circulation of the atmosphere may be taken to have in suspension the maximum amount of moisture, and, other things being equal, to approximate the saturation amount theoretically to be expected in air of a given pressure and at a given temperature.

WINDS.

Throughout California the general drift or movement of the atmosphere is to the southeast. Over the land, however, the direction and velocity of the wind are much modified by the trend of the mountain ranges and intervening valleys, as well as by diurnal and seasonal changes in temperature. Along the coast the northwest passage winds blow with remarkable regularity during the summer, and more or less constantly during the entire year. The diurnal changes in

temperature over the land cause the daily fluctuation in these winds. Ordinarily the wind from the ocean begins to blow inland during the forenoon and increases in force until late in the afternoon, when it gradually subsides. This is known as the sea breeze. During the night there is often a light breeze seaward, known as the land breeze. The sea and land breezes are but local modifications of the general drift of the atmosphere, and depend principally upon the difference in temperature between the land and the sea. In valleys that open toward the northwest and extend far inland between high mountain ranges, as the Salinas Valley, the sea winds are usually strong, and during the summer season often violent.

In the accompanying tables are given anemometer records of the total wind movement in miles per day at Salinas from January 1, 1902, to March 13, 1903, and at Kings City from January 1, 1902, to March 14, 1903. The greater part of this wind movement is up the Salinas Valley and takes place during the afternoon.

Total daily wind movement, in miles, at Salinas.

Date.	1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.
1902.											
January	------	------	------	------	------	------	59	134	91	130	140
February	115	111	118	107	114	128	199	215	139	103	181
March	------	------	159	127	161	217	161	228	197	201	144
April	206	183	201	160	122	104	308	131	151	147	180
May	207	174	174	156	125	242	172	157	169	124	211
June	247	281	213	203	184	180	180	174	170	204	----
July	------	------	------	------	------	------	------	------	104	145	133
August	192	242	197	186	166	186	176	195	191	191	190
September	142	169	171	236	132	173	151	172	157	143	146
October	202	185	161	172	92	139	181	120	130	167	162
November	125	131	163	120	133	112	133	225	462	355	104
December	134	115	148	167	96	109	66	149	131	214	146
1903.											
January	97	125	109	158	200	214	145	94	98	132	107
February	150	161	131	220	148	130	140	230	119	96	197
March	221	163	152	166	193	129	133	170	325	101	180

Total daily wind movement, in miles, at Salinas—Continued.

Date.	12.	13.	14.	15.	16.	17.	18.	19.	20.	21.
1902.										
January	182	112	90	59	103	123	132	123	99	100
February	156	124	255	349	212	177	115	150	162	218
March	131	180	243	180	149	53	240	149	137	149
April	153	129	138	127	168	161	160	175	223	147
May	135	197	141	154	162	210	257	304	187	175
June										
July	289	200	168	206	188	204	185	169	185	199
August	165	170	184	163	198	203	148	191	222	140
September	157	153	125	165	128	146	140	164	236	165
October	145	114	223	115	159	129	157	131	123	41
November	105	115	94	89	127	135	90	175	84	132
December	97	52	100	156	257	107	112	128	90	140
1903.										
January	194	187	149	145	178	106	129	85	106	91
February	100	160	159	139	133	142	167	164	183	160
March	141	241	257							

Date.	22.	23.	24.	25.	26.	27.	28.	29.	30.	31.
1902.										
January	96	174	181	240	255	80	96	102	239	48
February	280									
March	210	203	181	164	107	100	120	120	146	148
April	226	88	188	128	157	164	163	223	50	
May	188	137	150	179	243	138	209	152	227	284
June										
July	173	161	177	236	271	203	174	135	211	209
August	177	183	205	162	165	149	143	179	152	150
September	163	181	179	171	171	151	205	158	180	
October	286	212	179	125	116	99	85	126	119	131
November	139	144	121	111	131	94	91	101	131	
December	235	128	199	129	130	95	65	135	150	106
1903.										
January	122	106	125	105	174	222	206	93	109	221
February	107	113	121	98	129	122	143			
March										

Total daily wind movement, in miles, at Kings City.

Date.	1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.
1902.											
January											
February	126	110	122	136	159	153	170	190	141	125	124
March	160	312	180	151	137	295	90	355	178	207	221
April	271	181	109	166	60	220	304	199	283	262	262
May	228	299	264	197	98	197	183	332	253	578	00
June	302	331	348	344	267	301	312	209	231	238	324
July	300	243	240	417	244	296	243	229	239	308	214
August	306	289	234	271	245	262	242	282	294	326	367
September	189	210	257	216	255	229	244	208	248	237	206
October	490	482	86								
November	178	63	228	222	169	146	131	264	341	370	136
December	147	162	222	86	123	145	94	147	202	441	187
1903.											
January	134	136	128	177	158	140	149	102	122	102	140
February	128	124	236	79	118	91	80	267	163	141	162
March	151	150	299	123	167	121	99	258	241	121	192

Date.	12.	13.	14.	15.	16.	17.	18.	19.	20.	21.
1902.										
January										
February	214	162	224	299	200	331	250	230	154	213
March	192	292	240	282	208	95	282	194	162	322
April	337	237	208	169	219	285	233	324	313	128
May	247	184	225	240	291	430	395	464	289	234
June	306	330	396	359	284	280	216	271	258	267
July	210	270	259	381	279	350	181	283	307	253
August	287	175	167	235	321	244	216	246	206	212
September	216	246	255	243	203	177	192	184	264	169
October		100	118	204	115	225	119	161	131	227
November	156	123	107	165	112	180	190	146	127	162
December	76	103	140	132	194	69	53	99	64	77
1903.										
January	95	111	107	186	135	155	128	172	243	132
February	193	198	127	91	123	119	122	147	145	124
March	180	307	266							

Total daily wind movement, in miles, at Kings City—Continued.

Date.	22.	23.	24.	25.	26.	27.	28.	29.	30.	31
1902.										
January					374	118	101	105	157	124
February	323	143	397	240	320	192	146			
March	257	311	163	215	152	173	156	197	128	190
April	213	157	198	217	322	160	204	156	214	.
May	173	207	254	262	200	243	323	367	349	335
June	243	250	360	273	253	405	338	238	288	...
July	228	210	236	283	210	305	323	304	286	337
August	239	277	268	214	231	204	213	225	237	219
September	140	163	276	314	270	142	289	189	148	...
October	210	276	280	132	112	103	171	203	175	270
November	88	115	91	78	110	126	153	204	196	....
December	99	98	98	150	195	111	209	113	100	125
1903.										
January	147	109	130	162	187	312	320	175	197	56
February	124	106	236	214	149	109	153	.....	.....	.....
March	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

RAINFALL.

When the passage winds reach the coast they sometimes, under proper conditions, precipitate a portion of their moisture. One of these conditions is cooling of the air by contact with a cool land area or by being forced to a great altitude in passing over mountain ranges, for the cold summits of mountains are effective condensers of moisture. Such conditions do not, however, occur along the coast of Monterey County, for the general movement of the air currents there is from the northwest to the southeast, parallel with and not across the mountain ranges. In addition to this the air currents in their southeastward drift are continually passing into a warmer region, where the temperature of the air is raised and its capacity for absorbing moisture increased. Thus the passage winds, although moist and cool on the coast, become dry in the interior and abstract moisture by evaporation from every stream and lake, from the soil, and from all vegetation. These northwest winds bring no moisture to the land along the coast south of San Francisco.

The above-described climatic conditions are practically constant, recurring from day to day throughout the year unless modified by general atmospheric disturbances accompanying regional storms. As is well known, in cyclonic storms of vast extent the storm center is an area of low barometer toward which the winds blow from every

direction. The path of these storms shifts with the seasons, moving southward during the autumn and reaching the coast of Washington and Oregon in October or November. Many pass eastward without bringing rain to central or southern California; others, however, are deflected southward by atmospheric conditions east of the Cascade Mountains and the Sierra Nevada and pass over California, Nevada, and Arizona. These storms are preceded by southeast, south, or southwest winds along the coast regions of California, or toward the storm center in the northern part of the State. The general direction of these storm winds is nearly at right angles to the general drift of the passage winds and across the trend of the mountain ranges of the State. This marked change in the direction of the wind is an important factor in the rainfall on the Pacific coast. These moist southerly winds pass from a warm to a cooler region and are cooled sufficiently to cause precipitation, thus giving rain over sea and land.

These moisture-laden storm winds from the Pacific, on encountering the mountain ranges along the coast and in the interior, are forced to rise to a considerable altitude, where they are still further cooled and precipitate a large part of their moisture, leaving the regions to the east of the mountain ranges dry and barren.

Aridity, or rather the unequal distribution of moisture, is largely the result of topography or inequalities of land surface. If the land were perfectly flat, it is probable that the winds, meeting with no obstructions, would distribute the rains with considerable uniformity in broad bands approximately parallel to the equator; but the relatively thin layer of dense atmosphere surrounding the globe is disturbed in its uniform flow by the lofty mountain masses which traverse the continents. The atmosphere surrounding the earth extends outward many miles, but it is the layer, a mile or two in thickness, resting immediately upon the surface, and relatively dense, within which occur the changes or disturbances that make up what we know as "weather." The movements of the air above this thin layer concern us little, but the behavior of the clouds and winds near the surface of the ground brings success or failure to the farmer, and affects more or less directly other industries, and even health.[a]

The marked influence of topography on the distribution of rainfall is strikingly shown in fig. 1. This diagram shows graphically the increase in rainfall due to difference in altitude on the west side of the Sierra Nevada and the no less pronounced decrease on the east side, indicating how completely the moisture-laden winds have been robbed of their moisture by the cold mountain range. This diagram consists of a profile or section drawn along a line from Farallon Islands through San Francisco, across the Sacramento Valley and Sierra Nevada, and out on the Great Basin as far as Wadsworth. The various stations along this profile and their altitude above sea level are shown. The height of the small black columns represents the average annual rainfall in inches at each station.

[a] Newell, F. H., Irrigation, p. 16.

At the Farallon light-house on Farallon Island, 30 miles west of San Francisco, the average rainfall is 18.44 inches. This may be taken as the probable average along the coast at this latitude, where the precipitation is not influenced by adjacent mountain masses. At San Francisco the annual rainfall is 22.77 inches, the increase being due to the influence of the Coast Range. Toward the east, in the Sacramento Valley, the annual rainfall is about the same as over the ocean, being 19.78 inches at Fairfield and 19.60 inches at Sacramento. Folsom is at the foot of the Sierra Nevada, at an altitude of 180 feet, and has an annual rainfall of 23.81 inches. On the western slope the rainfall increases with increase in altitude, but not uniformly. The rainfall at Auburn is 33.06 inches, at Colfax, 47.02 inches, and at Cisco, a short distance west of the summit, 51.18 inches. The increase from Folsom to Colfax is at the rate of some-

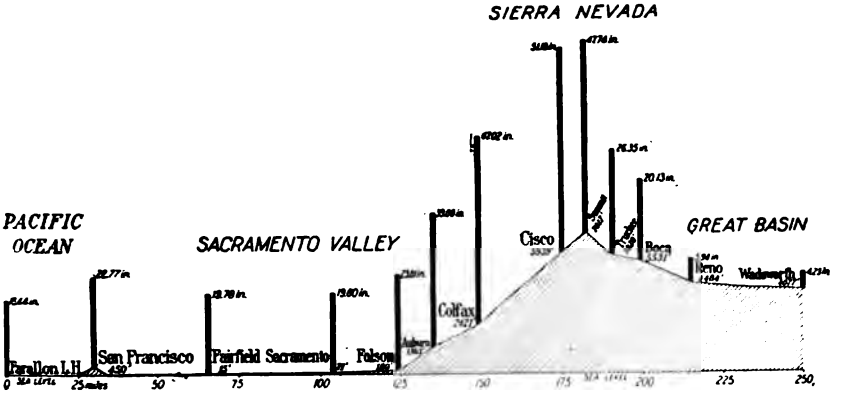


FIG. 1.—Diagram showing unequal distribution of rainfall in California and Nevada, the result of topographic features.

what more than 1 inch per 100 feet rise in elevation, while from Colfax to Cisco the increase is at the rate of but a little more than 0.1 inch per 100 feet rise in elevation, or about one-tenth of the increase below. At Summit, on the crest of the range, the rainfall is 47.74 inches, or 3.44 inches less than at Cisco, although Summit is 1,028 feet higher. The sudden decrease in rainfall on the lee side of the range is strikingly shown in this diagram. At Summit the rainfall is 47.74 inches and at Truckee 26.35 inches. The difference in altitude of these two points is 1,197 feet and the actual decrease in rainfall is 21.39 inches, a trifle over $1\frac{3}{4}$ inches decrease per 100 feet drop in altitude. This rate of decrease is many times greater than the rate of increase on the opposite side of the range at the same altitude.

It is believed that the following general conclusions can be justly drawn from a study of this section:

(1) The average rainfall over the sea in the latitude of San Francisco is about 18.50 inches.

(2) On the southwest slopes of mountain ranges exposed to the storm winds the increase in rainfall is at the rate of approximately 1 inch per 100 feet rise up to an altitude of 2,500 feet.

(3) Between about 2,500 and 6,000 feet there is still an increase in the amount of rainfall, but at a much lower rate.

(4) Above 6,000 feet there is a slight decrease in the rainfall.

(5) The first high cold land passed over by the winds cools the atmosphere and condenses a large part of the moisture, leaving less or little for the land beyond.

The available rainfall records in the Salinas Valley and adjacent regions are given in the accompanying table. When these records are examined it is seen that there is a great difference in the amount of rainfall at different points—as at Mansfield and at San Ardo, for example. Fig. 2 is a graphic illustration of these variations. The section is similar to the one in fig. 1, and extends from the Pacific Ocean across the Santa Lucia Range, the Salinas Valley, and the

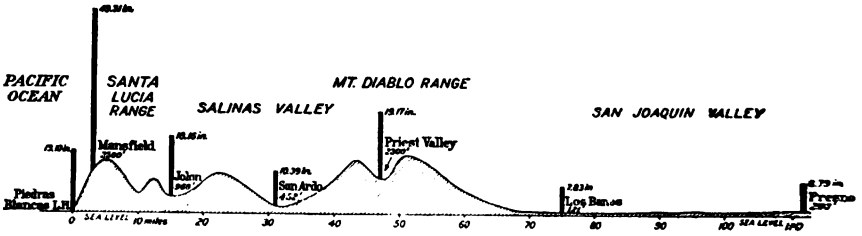


FIG. 2.—Diagram showing unequal distribution of rainfall in the Coast Range of California, the result of topography.

Mount Diablo Range to Fresno, on the east side of the San Joaquin Valley. The Santa Lucia Range, the most elevated land mass on the coast of central California, fronts the Pacific Ocean for more than 100 miles, exposing a bold, rugged system of peaks and ridges to the full force of the storm winds that sweep up from the southwest. Over this range these moisture-laden winds, in their passage inland, are forced to rise in altitude from 3,500 to 6,000 feet. This range, from its position and altitude, should be one of the dominant factors in the distribution of rainfall in this region, and an examination of the rainfall diagram given in fig. 2 shows that this is the case.

At Piedras Blancas light-house, near sea level, the average annual rainfall is 19.10 inches. At Point Sur light-house it is 18.11 inches. The rainfall at sea level here is apparently about the same as at Farallon light-house. In passing eastward along the line of this profile the next station is Mansfield, at an altitude of about 2,500 feet, and but 3 or 4 miles from the ocean. The average rainfall here is 49.31 inches, or more than two and one-half times greater than at sea level. The rate of increase with rise in elevation is somewhat greater here

Seasonal rainfall, in inches, in the Salinas Valley and the adjacent coast region.

[This table shows the total rainfall for twelve months, beginning September 1 and ending August 31.]

	Stations.																	
	Santa Cruz.[a]	Pajaro.[a]	Salinas.[a]	Soledad.[a]	San Ardo.[a]	San Miguel.[a]	Paso Robles.[a]	Santa Margarita.[a]	Jolon.[b]	Los Vaqueros.[c]	Monterey.[a]	Point Sur light-house.[d]	Piedras Blancas light-house.[d]	San Luis Obispo.[a]	Priest Valley.[e]	Simmler.[f]	Goodwin.[g]	Mansfield.[h]	Parkfield.[i]
Elevation in feet	25	28	44	183	452	616	723	996	960	950	0	0	0	270	2,300	.....	.....	2,500	.....
Season—																			
1868–69	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	[j]18.15	.....	.....	.....	.....	.....	.....	.....	.....
1869–70	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	13.59	.....	.....	12.03	.....	.....	.....	.....	.....
1870–71	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	[j]10.45	.....	.....	12.97	.....	.....	.....	.....	.....
1871–72	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	[j]21.56	.....	.....	26.02	.....	.....	.....	.....	.....
1872–73	.....	.....	13.45	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	12.79	.....	.....	.....	.....	.....
1873–74	.....	18.49	18.61	6.35	.....	.....	.....	.....	.....	.....	.....	.....	.....	20.52	.....	.....	.....	.....	.....
1874–75	.....	12.41	8.05	5.01	.....	.....	.....	.....	.....	.....	.....	.....	.....	19.69	.....	.....	.....	.....	.....
1875–76	.....	26.17	19.16	15.34	.....	.....	.....	.....	.....	.....	.....	.....	.....	30.12	.....	.....	.....	.....	.....
1876–77	.....	5.53	3.90	2.65	.....	.....	.....	.....	.....	.....	.....	.....	.....	8.15	.....	.....	.....	.....	.....
1877–78	36.41	28.01	23.75	12.22	.....	.....	.....	.....	.....	.....	32.02	.....	.....	30.60	.....	.....	.....	.....	.....
1878–79	22.06	17.37	9.32	4.15	.....	.....	.....	.....	.....	.....	11.89	.....	.....	11.66	.....	.....	.....	.....	.....
1879–80	18.22	16.54	13.33	7.38	.....	.....	.....	.....	.....	.....	[j]16.78	.....	.....	25.81	.....	.....	.....	.....	.....
1880–81	29.34	19.91	12.43	6.78	.....	.....	.....	.....	.....	.....	13.91	.....	.....	23.69	.....	.....	.....	.....	.....
1881–82	22.83	18.89	12.86	9.74	.....	.....	.....	.....	.....	.....	15.16	.....	.....	17.03	.....	.....	.....	.....	.....
1882–83	19.62	16.88	11.21	8.64	.....	.....	.....	.....	15.90	.....	16.64	.....	.....	16.83	.....	.....	.....	.....	.....
1883–84	30.74	23.37	19.44	16.36	.....	.....	.....	.....	28.65	.....	21.38	.....	.....	42.40	.....	.....	.....	.....	.....
1884–85	16.40	12.91	9.30	5.79	.....	.....	.....	.....	11.75	.....	11.11	.....	.....	17.59	.....	.....	.....	.....	.....

[a] U. S. Weather Bureau and Southern Pacific Railway Company.
[b] T. T. Tidball.
[c] J. W. Leigh.
[d] U. S. Weather Bureau.
[e] A. J. Meyers and Mrs. M. Griffin.
[f] A. Hubbard.
[g] [Name of observer not known.]
[h] J. M. Krenkel.
[i] C. H. Royse.
[j] Broken record.

Seasonal rainfall, in inches, in the Salinas Valley and the adjacent coast region—Continued.

	Stations.																	
	Santa Cruz.	Pajaro.	Salinas.	Soledad.	San Ardo.	San Miguel.	Paso Robles.	Santa Margarita.	Jolon.	Los Vaqueros.	Monterey.	Point Sur light-house.	Piedras Blancas light-house.	San Luis Obispo.	Priest Valley.	Simmler.	Goodwin.	Mansfield.	Parkfield.
Elevation in feet	25	23	44	183	452	616	723	996	960	950	0	0	0	270	2,300	.....	.....	2,500	.....
Season—																			
1885-86	32.07	27.05	20.56	14.45	.....	.....	.....	.....	31.42	.....	18.50	.....	.....	29.30	.....	.....	.....	.....	.....
1886-87	17.17	12.85	9.88	6.74	7.92	[a] 9.86	[a] 10.72	.....	12.42	.....	9.16	.....	.....	16.56	.....	.....	.....	.....	.....
1887-88	22.91	15.71	12.70	6.15	9.91	11.03	14.30	.....	16.77	.....	12.78	.....	.....	19.33	.....	.....	.....	.....	.....
1888-89	23.35	15.76	11.63	10.68	14.10	11.89	15.84	.....	24.74	.....	12.86	.....	.....	19.54	.....	.....	.....	.....	15.81
1889-90	54.68	39.88	28.44	18.67	23.27	20.13	29.85	49.79	36.94	.....	29.32	.....	.....	.....	.....	.....	.....	.....	21.70
1890-91	19.46	18.98	12.57	7.50	8.87	11.33	16.42	23.77	14.97	[a] 5.00	12.30	.....	.....	.....	.....	.....	.....	.....	11.36
1891-92	24.22	17.97	12.93	8.15	5.83	7.42	11.98	20.52	10.66	[a] 5.00	11.90	12.77	14.02	.....	.....	12.19	.....	.....	8.87
1892-93	33.42	28.93	18.03	12.02	12.92	13.63	22.65	41.68	27.23	26.76	.....	20.98	20.89	[a] 30.85	.....	6.75	.....	.....	13.15
1893-94	26.86	[a] 17.46	13.70	5.20	3.85	4.03	6.83	12.95	7.27	[a] 6.00	.....	14.71	17.35	.....	.....	5.17	.....	.....	4.66
1894-95	36.85	26.10	17.04	10.84	10.92	12.79	17.09	34.52	19.58	17.88	.....	23.49	19.91	25.10	.....	12.34	.....	.....	12.34
1895-96	21.44	18.45	12.78	9.98	14.59	11.12	17.94	26.55	17.96	7.10	18.79	18.26	18.64	[a] 17.74	.....	9.37	.....	[a] 57.69	11.17
1896-97	27.95	19.90	14.93	9.60	[a] 9.22	10.82	16.83	25.58	16.85	[a] 16.18	14.79	18.00	24.36	20.59	.....	10.33	.....	[a] 56.94	11.26
1897-98	12.49	10.15	8.50	3.47	3.65	3.47	4.75	8.44	5.80	8.57	6.95	9.54	8.24	7.20	.....	1.44	.....	[a] 19.75	4.12
1898-99	25.09	22.60	10.55	5.39	7.40	8.15	11.53	19.19	12.00	12.98	10.42	20.14	26.02	17.33	12.16	2.70	.....	[a] 48.75	6.95
1899-1900	28.38	21.50	11.68	6.68	7.73	8.73	11.66	19.62	11.90	10.38	12.38	16.61	19.27	17.21	15.70	5.70	.....	[a] 42.75	10.96
1900-1901	28.27	24.92	15.63	12.39	15.70	15.18	22.80	48.82	26.12	24.46	16.00	23.19	22.21	31.40	31.73	14.15	.....	[a] 69.45	14.49
1901-2	29.35	18.98	10.58	8.74	10.37	9.08	12.75	29.80	14.77	20.61	14.98	21.47	19.23	21.78	17.11	.....	.....	49.81	.....
Mean	26.31	19.61	13.70	8.93	10.39	10.54	15.25	27.78	18.16	16.09	15.53	18.11	19.10	20.64	19.17	8.01	5.00	49.81	11.29

[a] Broken record.

than in the Sierra Nevada. There are no records at a greater elevation in this range, but the precipitation undoubtedly increases with the altitude.

The next station on this section is at Jolon, 960 feet above sea level, where the average rainfall is 18.16 inches. To the northeast, in the broad, low Salinas Valley at San Ardo, the rainfall is still less, being but 10.39 inches. On the northeast side of the Salinas Valley the Mount Diablo Range rises to an average altitude of 3,500 to 4,000 feet, and there is a noticeable increase in the rainfall over this region. At Priest Valley, altitude 2,300 feet, the annual rainfall is 19.17 inches. This is, however, but a four years' record. In passing northeastward to the San Joaquin Valley, the rainfall diminishes, being 7.83 inches at Los Banos and 8.79 inches at Fresno. If similar sections should be drawn either to the north or south of this the same distribution of rainfall would be noted.

At Salinas the average annual rainfall is 13.70 inches, and there is a material increase to the northwest, there being 19.61 inches at Pajaro and 26.31 inches at Santa Cruz, these two stations being beyond the influence of the Santa Lucia Range. At Monterey the average rainfall is 15.53 inches.

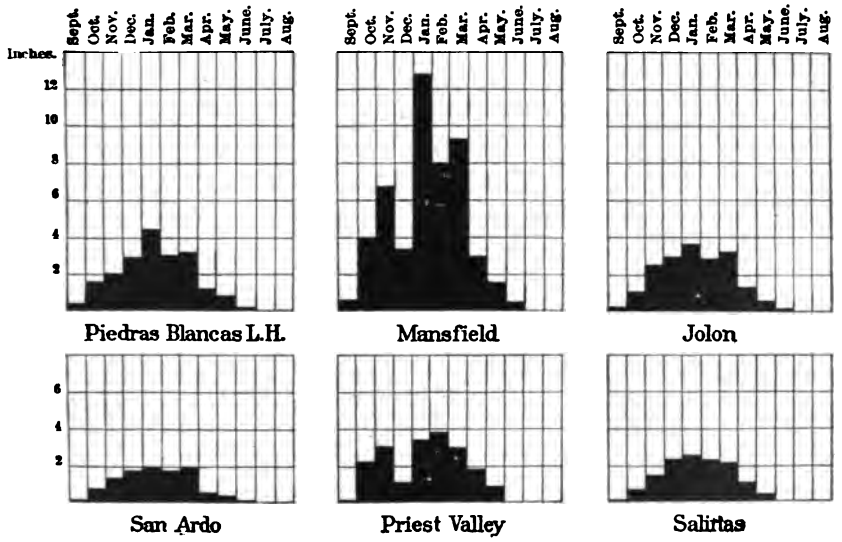


FIG. 8.—Diagram showing monthly distribution of rainfall in the Salinas Valley.

In passing up the Salinas Valley the rainfall is found to decrease as far as Soledad, where it is but 8.94 inches. To the southeast there is a steady increase, the records showing 10.39 inches at San Ardo, 10.54 inches at San Miguel, 15.25 inches at Paso Robles, and 27.78 inches at Santa Margarita.

In the southeast corner of Monterey County, at Parkfield, the rainfall is 11.29 inches. Over the Carriso Plain, probably the most arid region in the Coast Range, the rainfall is light, being but 8.01 inches at Simmler and about 5 inches at Goodwin.

The average monthly precipitation at six stations in the Salinas Valley is shown in fig. 3. The height of each of the small black columns represents the average amount of rain for the corresponding month. This diagram shows graphically the contrast between the distribution of the rainfall in summer and winter.

To summarize, it may be said that the distribution of rainfall in Monterey County depends upon the topography. The region may be divided into three topographic zones or belts in which rainfall conditions are similar. The most western belt is the Santa Lucia Range, the central is the Salinas Valley, with the mesa lands on the northeast side, and the eastern is the Mount Diablo Range.

In the Santa Lucia Range the rainfall varies from about 18 inches at sea level to 50 inches along the summit of the range, decreasing on the northeastern slope to about 18 inches at an altitude of 1,000 feet. In the upper Salinas Valley the rainfall is about 10 inches, decreasing gradually toward the southeast. From the incomplete records in the Mount Diablo Range it seems probable that the average rainfall is from 20 to 25 inches in Monterey County.

HYDROGRAPHY.

SALINAS RIVER.

The main trunk stream of the Salinas Valley is Salinas River, which, with its tributaries, drains an area of 4,780 square miles. (See Pl. I.) The drainage basin is 150 miles long, extending from southern San Luis Obispo County to the Bay of Monterey, in Monterey County. The western tributaries of the Salinas have their sources in the elevated, well-watered Santa Lucia Range, while its eastern tributaries drain the low mesa lands and the western slopes of the Mount Diablo Range. The river drains all the area lying between the Santa Lucia Range on the southwest and the Gabilan and Mount Diablo ranges on the northeast except the inclosed basin east of San Juan Creek known as the Carriso Plain.

The following table shows the areas of the drainage basins of the river and its tributaries:

Areas of drainage basins of Salinas River and its tributaries.

	Sq. miles.
Total drainage of the Salinas River	4,780
Drainage of Salinas River above gaging station near Salinas	4,084
Drainage of Salinas River above Estrella Creek	591
Total drainage of Nacimiento River	380
Drainage of Nacimiento River above gaging station near Bryson	171
Total drainage of San Antonio River	342
Drainage of San Antonio River above Pinkerton dam site	322
Drainage of San Antonio River above gaging station near Jolon	161
Total drainage area of Arroyo Seco above Arroyo Seco canal No. 3	218
Drainage area of Arroyo Seco above gaging station at Pettitt ranch	215
Drainage area of Arroyo Seco above Currier dam site	184
Drainage area of Arroyo Seco above Foster dam site	161
Total drainage area of Los Vaqueros Creek above Pettitt dam site	28
Drainage area of Los Vaqueros Creek above Leigh dam site	22
Drainage area of Estrella Creek	924
Drainage area of Carriso Plain	470
Total drainage area of San Lorenzo Creek above Mathews dam site	235
Drainage area of Chalone Creek	128

These areas were determined by planimeter measurements from the Monterey and San Luis Obispo atlas sheets.

From San Miguel, in San Luis Obispo County, northwest to Wunpost, in Monterey County, the Salinas flows in a deep, wide canyon, which it has eroded several hundred feet below the level of the surrounding mesas. At Wunpost the mesa recedes on the northeast side and the river valley widens out somewhat into a long, narrow, terraced alluvial plain, with a width ranging from 1 mile at the upper end to 4 or 5 miles at Soledad. From Soledad to the Bay of Monterey the alluvial plain and bordering detritus slopes are from 6 to 10 miles wide, with terraces far less pronounced than those in the upper portions of the valley. Each of the terraces is an ancient flood plain of the stream. For this distance the stream flows in a broad, sandy channel from 500 to 1,500 feet wide, into which the waters sink in summer to reappear again at some point below. From Soledad to the ocean the stream is perennial, being fed by these underground waters. When wet the fine sand in the river bed becomes quicksand, making the stream dangerous to ford, especially in winter. The photographic views reproduced in Pl. III, and Pl. IV, *A*, show the general character of the stream during the dry season.

The large volume of water discharged by this stream during the rainy season has enabled it to trench its channel to a depth considerably below the alluvial plains on both sides of the river. For the same reason the grade of the stream is very slight, being but 1.33 feet per mile from the ocean to the gaging station at the Monterey road bridge; 5.5 feet per mile from the Monterey road bridge to Soledad and 6.5 feet per mile from Soledad to Bradley, where the bed of the stream is at an altitude of about 500 feet. These grades are approxi-

mate and are given merely to show the low gradient of this, the main drainage line of the region.

This stream is torrential in character, having very large flood discharge during the rainy season and being practically dry during the summer except in its lower portion. The winter flow is probably at all times sufficient for the needs of winter and spring irrigation.

That portion of the drainage basin of the Salinas that lies south of San Miguel, comprising 591 square miles, is on the northeast slope of the Santa Lucia Range, in the region of heaviest rainfall, and the run-off here is undoubtedly large.

A record of the discharge of the main stream was kept at the Monterey road bridge from January 8, 1900, to July 31, 1901. From these observations the following table has been prepared.

Estimated monthly discharge of Salinas River 4 miles south of Salinas.

[Drainage area, 4,084 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1900.						
January 8–31	2,700	223	848	40,368	0.31	0.19
February	223	40	105	5,831	.03	.03
March	223	25	73	4,489	.02	.02
April	32	19	22	1,309	.01	.01
May	19	17	17	1,045	.00	.00
June	17	15	16	952	.00	.00
July	10	6	8	492	.00	.00
August	8	6	7	430	.00	.00
September	8	2	6	357	.00	.00
October	4	1	2	123	.00	.00
November	33,600	0	2,413	143,583	.59	.66
December	1,050	82	295	18,139	.07	.08
1901.						
January	35,162	380	4,921	302,580	1.21	1.39
February	20,927	540	4,131	231,701	1.02	1.06
March	1,772	430	1,063	65,361	.26	.30
April	405	160	270	16,066	.07	.07
May	2,012	160	533	32,773	.13	.15
June	160	30	56	3,332	.01	.01
July	30	25	27	1,660	.01	.01

NOTE.—Gage heights and discharge measurements for 1901 are given in Water-Supply Paper No. 66, page 154.

A. SALINAS RIVER ABOVE GAGING STATION AT MONTEREY ROAD BRIDGE, NEAR SALINAS.

Shows quicksand in bed of stream.

B. SALINAS RIVER NEAR KINGS CITY.

Shows drifted sand blown upstream by the prevailing northwest wind.

ESTRELLA CREEK.

The most eastern branch of Salinas River is Estrella Creek (see Pl. I). At a point 20 miles southeast of its junction with the Salinas the two branches of the Estrolla are known as San Juan River and Cholame Creek. The San Juan drains the northeastern slopes of the San Jose Range and Cholame Creek drains the southwestern slope of the Mount Diablo Range. Between these two mountain ranges lie the elevated mesa lands of the upper Salinas Valley. This mesa region passes over into the Carriso Plain, the waters of which, however, do not flow to the sea, but toward a salt flat or salt lake which fills the lower portion of the plain. Nothing very definite is known regarding the average rainfall in this region (see table on p. 43), but it is certain that it is very light, probably less than 10 inches, over the mesa regions, which are cut off from the storm winds by the intervening Santa Lucia and San Jose ranges. It is probable that the greater portion of the water in this stream is derived from rainfall in the surrounding mountains. The area of the drainage basin of Estrella Creek is 924 square miles, and the area of the Carriso Plain is 470 square miles.

NACIMIENTO RIVER.

The Nacimiento is one of the large western tributaries of the Salinas, entering that stream between San Miguel and Bradley (see Pl. I). It has its sources among the highest peaks of the Santa Lucia Range, only 3 miles from the Pacific Ocean. It flows southeastward about 35 miles, receiving many small tributaries from the southwest, and emerges from the mountains after rounding the declining ridge of the Sierra de las Piedras. From this point to the Salinas it flows in a broad, sandy channel eroded in the mesa. This stream flows for 25 miles parallel with and but a few miles northeast of the crest of the range, in the belt of maximum precipitation (see fig. 2), where the average rainfall must be from 30 to 50 inches per annum. The fact that the precipitation here is great is shown by the character of the stream and the vegetation over this portion of the range. The total area of this watershed is 380 square miles, of which 300 square miles are mountainous, the remainder being rolling hills and river valley.

The Nacimiento watershed decreases in width from 7 miles near its head to 2 at its lower end. It is 40 miles long. The upper 7 miles is mountainous, as is all that portion lying between the river and the coast divide. The valley floor averages 1 mile in width, and extends downstream 10 miles from a point 7 miles below the head. The eastern wall of the valley is the western wall of the San Antonio. The lower 25 miles of the river lies in a narrow V-shaped canyon whose sides rise 300 to 1,500 feet. The mountain slopes are covered with brush and trees. The valley floor is shaded with white oaks of great size. For the most part the bed of the river is sandy and crosswise flat, and varies in width from 50 to 400 feet. In the open valley the banks range from 15 to 40 feet in height. There are good

dam sites at the mouth of Gabilan Creek, Piedras Altas, and Pebblestone Gorge, 7 miles below the Monterey County line, but there are no corresponding storage sites of value.[a]

The following table of discharge of Nacimiento River in March and April, 1901, has been prepared:

Estimated monthly discharge of Nacimiento River at Bryson post-office.

[Drainage area, 171 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1901.						
February 17 to 28	3,300	295	855	20,350	5.00	5.21
March	365	110	184	11,314	1.08	1.24
April	670	87	99	5,891	.58	.65

NOTE.—Gage heights and discharge measurements for 1901 are given in Water-Supply Paper No. 66, page 154.

This stream is torrential, with a great run-off during the rainy season, but it dwindles to a small creek by early summer. The character of the stream in June, 1902, is shown in Pl. IV, *B*.

SAN ANTONIO RIVER.

The drainage basin of San Antonio River joins that of the Nacimiento on the northeast, and the upper 120 square miles is in a similar mountain region (Pl. I). The area of the watershed above the Pinkerton reservoir site is 322 square miles. Of this area the mountain portion, 120 square miles, probably receives an average annual rainfall of 30 to 50 inches and the elevated valleys near Jolon as much as 18 inches. This stream leaves the mountains about 6 miles above Jolon, and from there to the Salinas flows in a broad, sandy channel through wide valleys. These valleys have an altitude of from 800 to 1,200 feet and are covered with scattering oak timber (see Pl. V).

The San Antonio drainage basin is 6 miles wide from the head for a distance of 24 miles, then for 5 miles it widens to 10 miles; in the next 5 miles it narrows to 3 miles, and this width continues for 15 miles to its mouth. The valley floor averages 1½ to 2 miles wide. The southwestern wall of the valley is 5,000 feet high at the upper end, and this height falls to a few hundred feet toward the mouth. The north wall of the valley, the Santa Lucia Range, is 3,000 to 6,000 feet high. The eastern wall decreases from north to south from 2,500 feet to a few hundred feet in elevation.

The valley floor extends to the San Antonio-Arroyo Seco divide, and the main stream has a comparatively light slope. The tributaries are short, and each for

[a] MS. report of Prof. Charles D. Marx.

A. SALINAS RIVER AT BRADLEY.

Shows broad-topped terrace on which the town is built, and the flat-topped hills of the mesa beyond.

B. NACIMIENTO RIVER AT GAGING STATION NEAR BRYSON POST-OFFICE.

A OAK GROVE IN SAN ANTONIO RIVER VALLEY, NORTH OF JOLON

B. SAN ANTONIO RIVER VALLEY SOUTHEAST OF JOLON.

the greater part of its course has a rapid fall. The upper part of the drainage basin is brush covered on steep slopes, and the valley floor is shaded by oaks. The slopes are bare on the east wall of the lower half of the basin. The valley furnishes no combination of large storage basin followed just downstream by a good dam site.[a]

A gaging station was established on this stream 1 mile south of Jolon, and a record of the discharge was kept from December 15, 1900, to April 30, 1901. From these observations the following table of discharge of San Antonio River has been prepared:

Estimated monthly discharge of San Antonio River at Jolon.

[Drainage area, 161 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1900.						
December 15 to 28	200	170	184	5,109	1.14	0.59
1901.						
January	3,700	64	523	32,158	3.25	3.75
February	1,700	110	509	28,268	3.17	3.31
March	400	84	171	10,514	1.06	1.22
April	740	39	83	4,939	.52	.58

NOTE.—Gage heights and discharge measurements for 1901 are given in Water-Supply Paper No. 66, page 154. Rating table on page 178 of same paper.

During the summer of 1900 this drainage basin was explored under the direction of Prof. Charles D. Marx to discover and survey any possible reservoirs. The only one discovered on this stream is on the Pleyto ranch, 6 miles southeast of Pleyto post-office, the dam site being in sec. 30, T. 24 S., R. 10 E. (see Pl. I), and about 12 miles above the junction of San Antonio River with Salinas River.

PINKERTON RESERVOIR.

A reconnaissance survey was made of this reservoir, known as the Pinkerton site, by Prof. Charles D. Marx. The contour map of the reservoir is shown in fig. 4, and a cross section of the canyon of San Antonio River at the dam site is shown in fig. 5. The highest contour of the reservoir is at an altitude of 950 feet, as determined by aneroid observations. The area of the drainage basin tributary to the reservoir in 322 square miles. The dam as designed was to be of sandstone rubble 50 feet high, 900 feet long on top, and 500 feet long at

[a] MS. report of Prof. Charles D. Marx.

the surface of the stream. The bed rock is sandstone and it is estimated to be 25 feet below the bed of the stream. Rock suitable for the construction of the dam can be quarried within 1,000 feet of the dam.

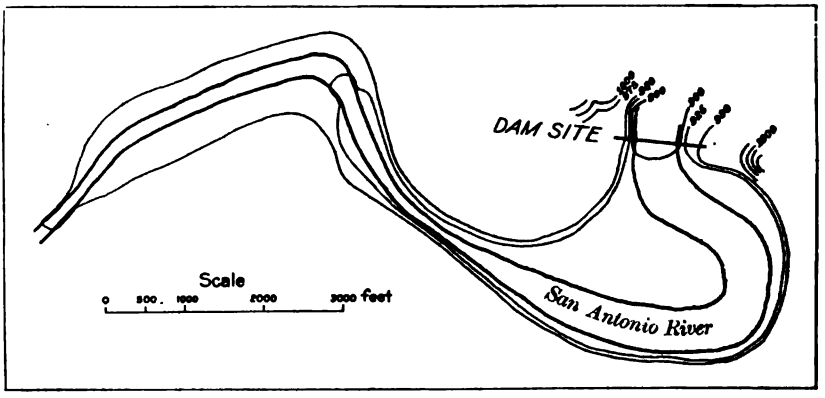


FIG. 4.—Reconnaissance map of Pinkerton reservoir site.

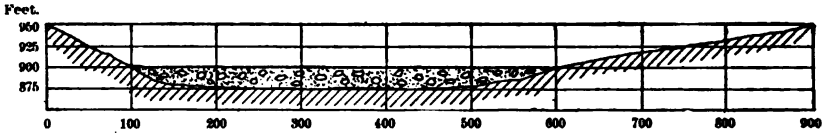


FIG. 5.—Cross section of canyon of San Antonio River at the Pinkerton dam site.

The following table shows the estimated storage capacity and the cost of storage at this location:

Estimated storage capacity and cost of storage at Pinkerton reservoir.[a]

Height of dam.	Total storage acre-feet.	Total cost of storage.	Cost per acre-foot.
25 feet	2,300	$92,000	$40.00
50 feet	8,200	210,500	26.00

ARROYO SECO.

The most northern tributary of Salinas River is the Arroyo Seco, which rises on the slopes of the highest portion of the Santa Lucia Range (Pl. I). The upper valleys of this stream are far back in the range, surrounded by high mountains. This portion of the range undoubtedly receives as great a rainfall as any other locality in this region. It does not seem unreasonable to suppose that the average annual rainfall is from 30 to 50 inches.

The drainage basin of the Arroyo Seco is almost entirely made up of sharp ridges and V-shaped canyons. The western portion is well covered with brush and trees of medium size. Toward the east this growth decreases, until at the

[a] MS. report of Prof. Charles D. Marx.

A. CANYON OF ARROYO SECO AT FOSTER DAM SITE, SHOWING STREAM TERRACE.

B. CANYON OF ARROYO SECO, 20 MILES WEST OF SOLEDAD, SHOWING STREAM TERRACES AND SANTA LUCIA MOUNTAINS.

Salinas the country is bare. The stream beds of this area fall rapidly, the Arroyo Seco rising at nearly 6,000 feet and emptying into the Salinas at an elevation of 170 feet.[a]

For 10 miles above its junction with the Salinas the channel of this stream is a broad wash of gravel and sand across the flat plain, into which the stream sinks in the dry season and from which it receives its name "Arroyo Seco." For the next 11 miles the stream flows in a deep terraced valley eroded into bituminous shales. The terraces form a most conspicuous feature (see Pl. VI and Pl. IX, *A*) and mark successive periods of uplift. They were cut into the bituminous shale or sandstone by the stream.

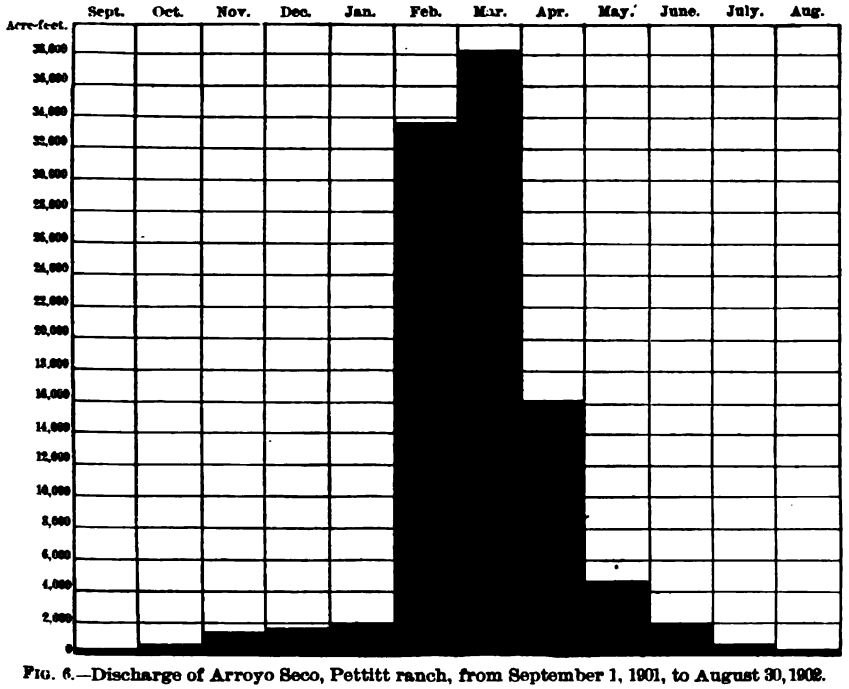


FIG. 6.—Discharge of Arroyo Seco, Pettitt ranch, from September 1, 1901, to August 30, 1902.

The present channel of Arroyo Seco in this 11-mile section is a deep trench or canyon, from 100 to 200 feet deep and 250 to 1,000 feet wide, sunk below the floor of the valley. The bottom of this canyon is covered with great washes of granite bowlders and sand. (See Pl. VII, *A*.) This débris conjoins with the floods of water that rush down from the mountains above to form the agencies that are now actively eroding the channel of this stream.

A record of the discharge of this stream was kept at Pettitt ranch during 1902 and a portion of 1901. From these observations the table on page 54 has been prepared. Fig. 6 is a graphic illustration of the discharge during the season of 1901–2.

[a] MS. report of Prof. Charles D. Marx.

Estimated monthly discharge of Arroyo Seco at Pettitt ranch.

[Drainage area, 215 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1901.						
January	4,500	105	888	54,601	4.13	4.76
February	2,860	160	931	51,705	4.33	4.51
March	610	105	246	15,126	1.14	1.32
April	2,500	95	195	11,603	.91	1.01
May			95	5,841	.44	.51
June			58	3,451	.27	.30
July			22	1,353	.10	.12
August			15	922	.07	.08
September			8	476	.04	.04
October			10	615	.05	.05
November	50	9	22	1,309	.10	.11
December	40	23	26	1,599	.12	.14
1902.						
January	40	25	28	1,722	.13	.15
February	2,860	25	605	33,600	2.81	2.93
March	2,320	185	620	38,122	2.88	2.32
April	1,650	145	270	16,066	1.26	1.41
May	105	55	74	4,550	.34	.39
June	55	15	32	1,904	.15	.17
July	15	7	12	738	.06	.07
August	7	3	4	246	.02	.02
September	3	0	0	0	0.00	0.00
October	55	0	9	553	.04	.05
November	750	11	57	3,392	.27	.30
December	145	25	54	3,320	.25	.29
1903.						
January	1,790	22	188	11,560	.87	1.00
February	970	120	313	17,383	1.46	1.52
March	3,080	92	471	28,961	2.19	2.52
April	3,210	120	483	28,740	2.25	2.51
May	120	56	85	5,226	.40	.46
June	56	32	46	2,737	.21	.23
July	32	13	25	1,537	.12	.14
August	22	6	10	615	.05	.06

NOTE.—Gage heights and discharge measurements for 1901 are given in Water-Supply Paper No. 66, page 156. Rating table on page 178, same paper.

During the summer of 1900 the drainage area of Arroyo Seco was explored by Prof. Charles D. Marx. Three reservoir sites were discovered on Arroyo Seco—The Pools site, the Foster site, and the Currier site (Pl. I). Two sites were also discovered on Los Vaqueros Creek, a tributary of Arroyo Seco—the Pettitt site and the Leigh site. The Foster and Currier reservoir sites were resurveyed by the writer in the summer of 1902. These reservoir sites will be described in detail.

THE POOLS RESERVOIR.

The Pools reservoir site is far back in the Santa Lucia Mountains, just below the junction of Arroyo Seco and Santa Lucia Creek (Pl. I).

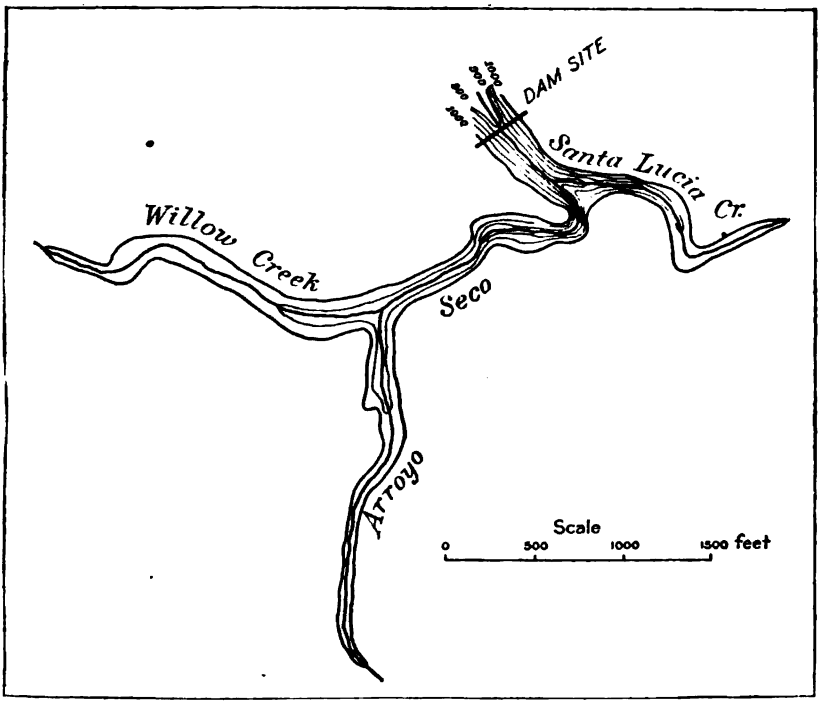


FIG. 7.—Reconnaissance map of The Pools reservoir site.

It is about 20 miles west of Soledad. A reconnaissance survey was made of this reservoir site by Prof. Charles D. Marx in the summer of 1900. A contour map of the reservoir forms fig. 7, and a cross section of the canyon of Arroyo Seco at the dam site is given in fig. 8. The highest contour of the reservoir is at an altitude of 1,000 feet, as determined by aneroid observations. The drainage area tributary to the reservoir is 74 square miles of steep mountain slopes covered with thick brush and trees.

The dam as designed was to be of granite rubble, with a maximum

height of 100 feet and 250 feet long on top. The bed rock is granite and the estimated depth to solid rock is 5 feet. The following table shows the storage capacity and cost of storage at this location.

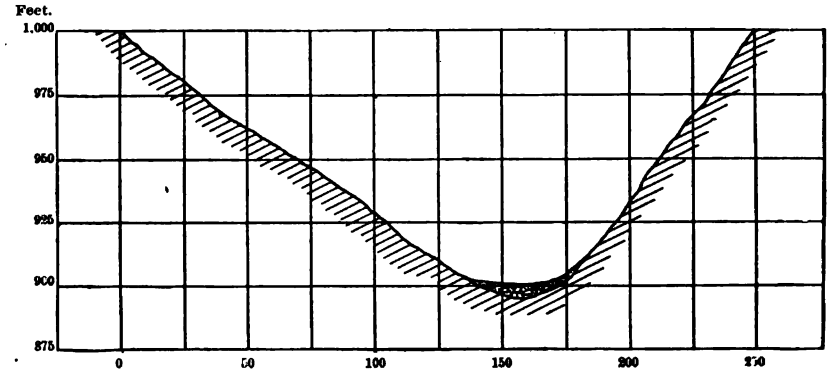


FIG. 8.—Cross section of canyon of Arroyo Seco at The Pools dam site.

Estimated storage capacity and cost of storage at The Pools reservoir.[a]

Height of dam.	Total storage in acre-feet.	Total cost.	Cost per acre-foot.
25 feet	30	$10,200	$340
50 feet	130	28,800	222
75 feet	310	61,800	195
100 feet	660	115,800	175

The reservoir is merely a steep, V-shaped canyon. The cost of storage, $175 per acre-foot, is prohibitive, and with the great distance, 20 miles from irrigable land, serves to condemn this location.

PETTITT RESERVOIR.

The Pettitt reservoir is in sec. 20, T. 19 S., R. 6 E., M. D. M., at the mouth of Los Vaqueros Creek. It is about 11 miles south of Soledad. (See Pl. I.) A reconnaissance survey was made of this reservoir site by Prof. Charles D. Marx in the summer of 1900. A contour

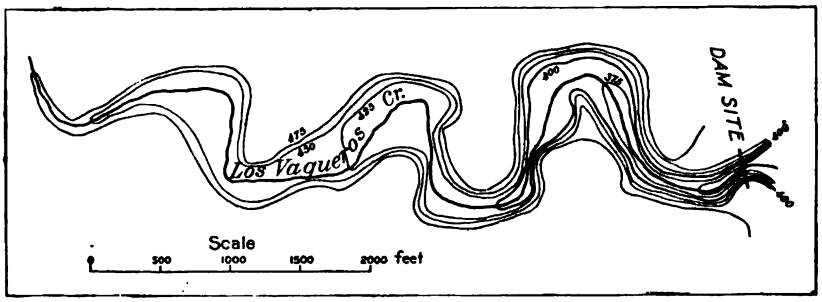


FIG. 9.—Reconnaissance map of Pettitt reservoir site.

[a] MS. report of Prof. Charles D. Marx.

map of the reservoir forms fig. 9, and a cross section of the canyon of Los Vaqueros Creek at the dam site is given in fig 10.

The highest contour of the reservoir has an altitude of 450 feet, as determined by aneroid observations. The area of the drainage basin tributary to the reservoir is 28 square miles. The dam as designed

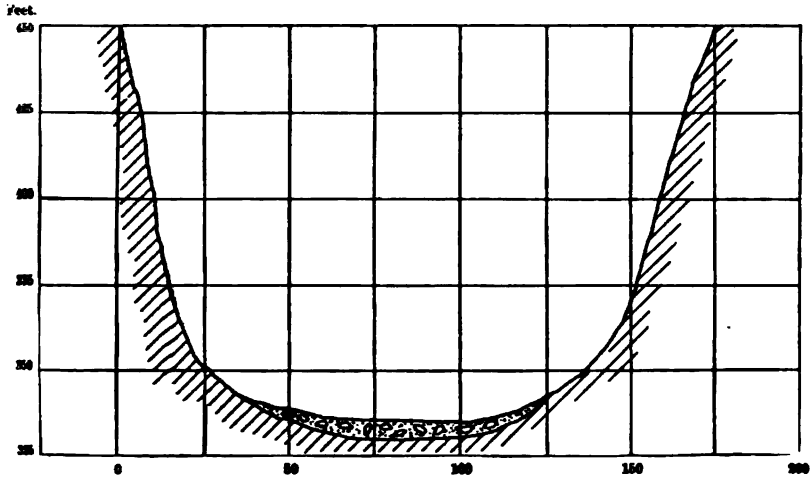


FIG. 10.—Cross section of Los Vaqueros Creek at Pettitt dam site.

was to be a concrete masonry structure 115 feet high and 175 feet long on top. The bed rock is shale, and the estimated depth to solid rock is 5 feet.

The following table shows the estimated storage capacity and cost of storage at this location.

Estimated storage capacity and cost of storage at Pettitt reservoir.[a]

Height of dam.	Total storage in acre-feet.	Total cost.	Cost per acre-foot.
25 feet	15	$9,000	$635
50 feet	110	27,000	245
75 feet	420	54,000	130
100 feet	1,040	99,000	95
115 feet	1,650	139,000	84

[a] MS. report of Prof. Charles D. Marx.

LEIGH RESERVOIR.

The Leigh reservoir site is in sec. 9, T. 20 S., R. 6 E., M. D. M., in Los Vaqueros Valley, 4 miles south of the Pettitt site (Pl. I).

A reconnaissance survey was made of this reservoir site by Prof. Charles D. Marx in the summer of 1900. A contour map of the reservoir forms fig. 11, and a cross section of the canyon of Los Vaqueros Creek at the dam site is given in fig. 12.

The highest contour of the reservoir is at an altitude of 975 feet, as determined by aneroid observations. The drainage area tributary to the reservoir is 22 square miles of steep mountain slopes covered with brush. The altitude of the area ranges from 900 feet at the dam site to almost 6,000 feet on the slopes of Santa Lucia Peak.

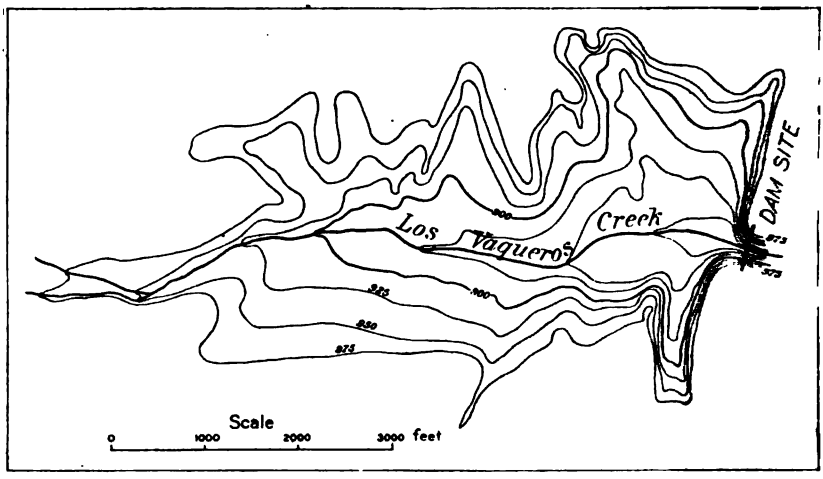


FIG. 11.—Reconnaissance map of Leigh reservoir site.

The dam as designed was to be of sandstone rubble, with a maximum height of 175 feet and a length of 400 feet on top. The bed rock is hard sandstone, and the estimated depth below the bed of the stream to solid rock is 15 feet. Sandstone suitable for the construc-

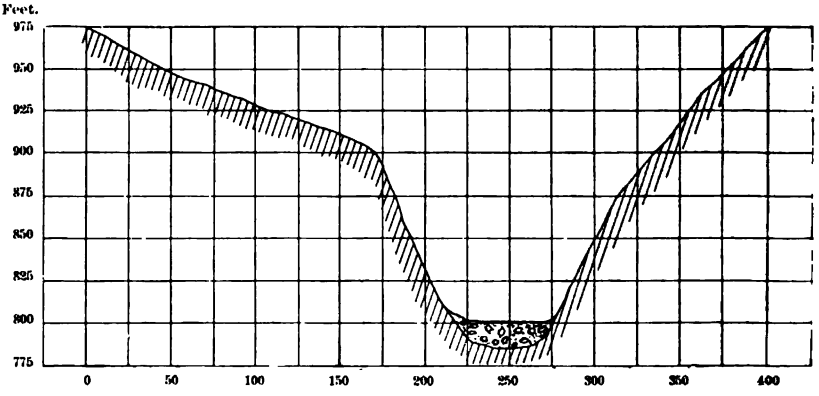


FIG. 12.—Cross section of Los Vaqueros Creek at Leigh dam site.

tion of the dam is available in the cliffs above, at both ends of the dam. The character of the canyon at the dam site is shown in Pl. VII, *B*. The irrigable lands are in the Salinas Valley, 10 miles below. The following table shows the estimated storage capacity and cost of storage at this location.

A. CHANNEL OF ARROYO SECO AT ABBOTTS, 20 MILES WEST OF SOLEDAD, SHOWING GRANITIC DETRITUS IN BED OF STREAM.

B. LEIGH DAM SITE ON LOS VAQUEROS CREEK, LOOKING DOWNSTREAM.

Estimated storage capacity and cost of storage at Leigh reservoir.[a]

Height of dam.	Total storage in acre-feet.	Total cost.	Cost per acre-foot.
25 feet	70	$8,600	$122.00
50 feet	600	21,600	36.00
75 feet	2,000	43,200	21.60
100 feet	4,600	77,000	16.60
125 feet	8,800	127,000	14.40
150 feet	14,800	194,000	13.00
175 feet	23,000	292,000	12.70

No measurements of the run-off from this drainage basin have been made. An examination of the discharge of Arroyo Seco, of which this stream is a tributary, shows that in 1901–2, a season of about average rainfall, the run-off was 0.78 second-foot per square mile (see table on p. 54). When this factor is used for computing the run-off of Los Vaqueros Creek a total run-off of approximately 10,000 acre-feet is given. As the rainfall at Los Vaqueros Valley is somewhat below the supposed average for this altitude, it is assumed that the run-off will be but 75 per cent of the amount computed as above, or approximately 7,500 acre-feet.

This will fill the reservoir to about the 925-foot contour. It seems probable that the drainage basin will supply this amount in a season of average rainfall, much more in a wet season, and but little in a dry season. It is believed that this will eventually be a valuable adjunct to the irrigation systems taking water from Arroyo Seco.

The dam should be built high enough to impound all the flood waters, the excess over 7,500 acre-feet being held over, if desirable, for the next year. The impounded water can be dropped into the channel of the creek and picked up again in Arroyo Seco at the head works of the canals without serious loss of water, so the only expense will be that of storage.

A cheaper dam, of the rock-fill type, could be constructed here, which would reduce the cost to probably one-third of the above estimate.

FOSTER RESERVOIR.

The proposed Foster reservoir is on Arroyo Seco, 12 miles southwest of Soledad in an air line and about 17 miles by the wagon road (Pl. II). The dam site is in the NW. ¼ sec. 23, T. 19 S., R. 5 E., M. D. M., and the reservoir site extends southwestward about 2½ miles. A survey of this location was made by the writer in the summer of 1902. A topographic map of the reservoir is given as fig. 13. The altitude of the stream at the dam site is 490 feet by aneroid observations.

[a] MS. report of Prof. Charles D. Marx.

The survey was made for a dam which would raise the water 110 feet above the bed of the stream, or to the 600-foot contour. Such a dam will impound 16,710 acre-feet of water. The table below shows the area and capacity of this reservoir at the various contours. Fig. 14

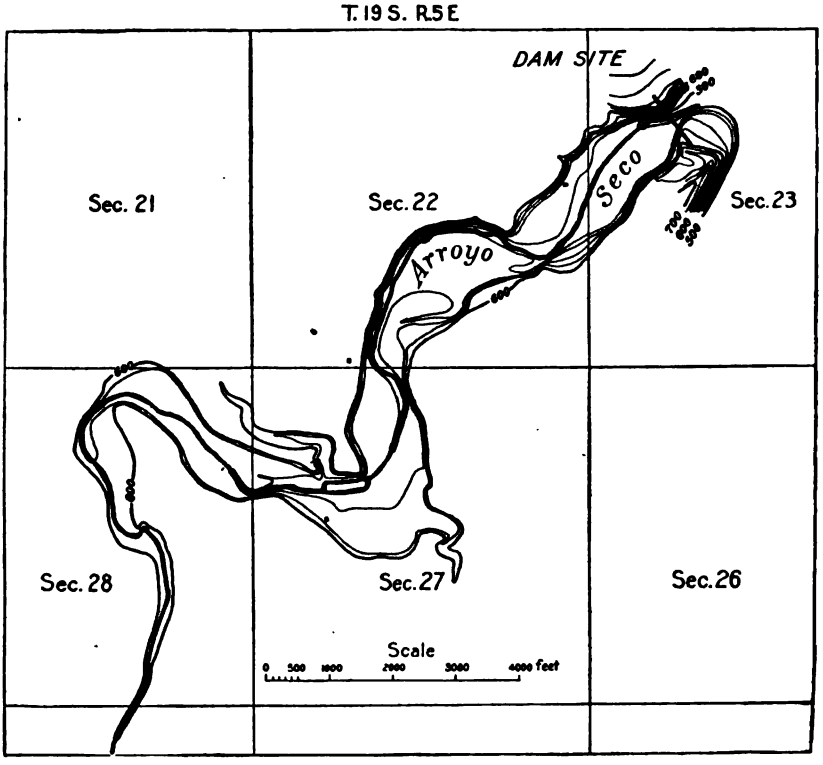


FIG. 18.—Foster reservoir site on Arroyo Seco.

Area and capacity of proposed Foster reservoir, Arroyo Seco.

Contour.	Area.	Capacity.
	Acres.	*Acre-feet.*
500	2.3	
510	22.2	122.6
520	50.6	486.6
530	80.1	1,140.1
540	117.8	2,129.1
550	149.6	3,466.1
560	192.8	5,178.1
570	252.1	7,402.1
580	292.5	10,125.1
590	329.0	13,232.1
600	366.6	16,710.1

shows the capacity curve of the reservoir, by means of which the amount of water stored can be ascertained for any depth. The drainage area tributary to this reservoir is 161 square miles. The table on page 54 shows the estimated monthly discharge of Arroyo Seco at the Pettitt ranch, 4 miles below. The discharge here will be practically the same as at the gaging station.

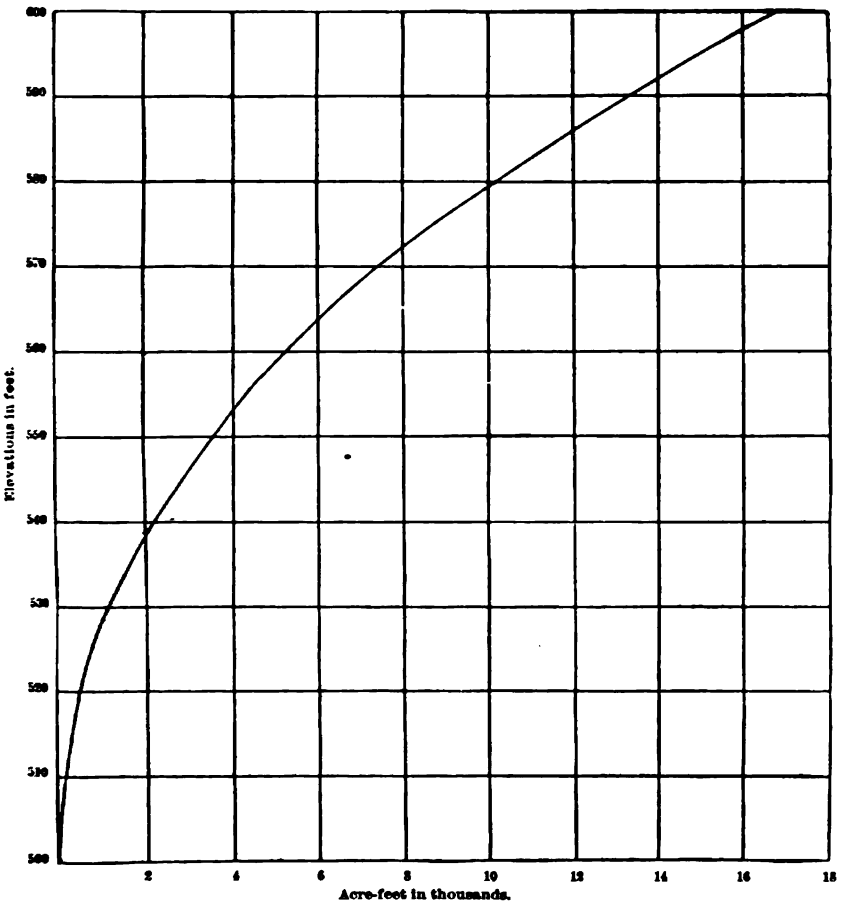


FIG. 14.—Capacity curve of Foster reservoir site, showing capacity, in thousands of acre-feet, with the water surface at various levels.

The reservoir site is situated in the terraced valley of Arroyo Seco, and is a long, narrow, steep-sided stream channel, the bottom of which is covered with washes of granitic bowlders and sand. Pl. VI, *A*, shows the general character of this channel looking upstream from below the dam site.

At the dam site the canyon has a width of 250 feet at the level of the stream and about 900 feet at the 615-foot contour, the top of the

dam. The rock at the dam site is the Monterey shale. The specific gravity of an ordinary specimen of the shale is 2 and the weight per cubic foot is about 125 pounds. These shales are hard and close textured where not weathered. In weathering they do not crumble into dust or sand as do ordinary rocks, but simply break into smaller fragments. At this point the shales are evenly stratified in thin beds which strike N. 75° W. and dip 40° SW. Rock is exposed in the bed of the stream at several places at the dam site, and it is probable that it is but a few feet to bed rock at any point. No borings have been made to bed rock, however. Pl. VIII, *A*, is a view of the north side of the canyon at the dam site, and Pl. VIII, *B*, is a view of the south side.

PROPOSED FOSTER DAM.

The dam at this point, as designed, is to be of the rock-fill type. Three different classes of material are available for the construction

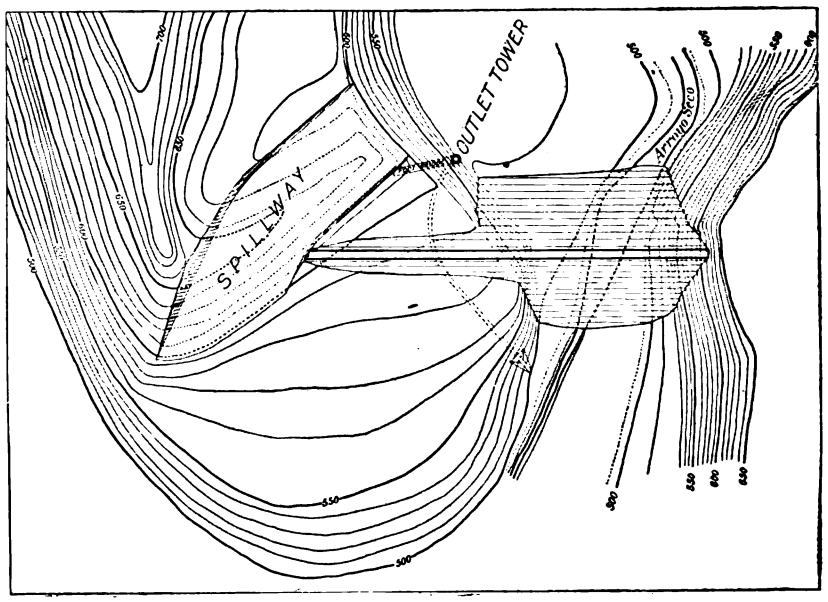


Fig. 15.—Foster dam site and spillway on Arroyo Seco.

of a dam here—(1) the Monterey shale on the south side of canyon; (2) the granitic bowlders and sand in the bed of the stream both above and below the dam site; (3) gravel, sand, and soil on the north side of the canyon. It was not considered safe to build an earthen or sand and gravel dam in the main channel of a torrential stream of this character, so the loose rock-fill type was chosen. It will be necessary to excavate a spillway in any case, and as located the excavation for the spillway will about make the fill for the dam. The relative position of the dam and spillway is shown in fig. 15.

A. NORTH ABUTMENT AT FOSTER DAM SITE.

B. SOUTH ABUTMENT AT FOSTER DAM SITE.

The dam will have a maximum height of 125 feet above the stream at the lower toe of the dam, as shown in fig. 16. The upstream slope is to be 1½ horizontal to 1 vertical and the downstream slope 1¼ horizontal to 1 vertical. The top width is to be 20 feet and the extreme length about 900 feet. The crest of the dam is to be 15 feet above the level of the spillway. This may seem excessive, but so little is known of the floods on this stream that it was not considered wise to make it less. In the future, if observations show that it is advisable, the level of the spillway can be easily raised.

The dam is to be made water-tight by a 1½-foot layer of asphaltic concrete laid on the upstream slope of the dam. This concrete is to be covered with 3½ feet of hand-laid rubble to protect it from the heat of the sun and weathering. The asphaltic concrete is to be made of sharp, angular broken rock and sharp sand, cemented with refined asphalt, properly tempered. The amount of asphalt should be reduced to the smallest amount that is required to secure a water-tight layer.

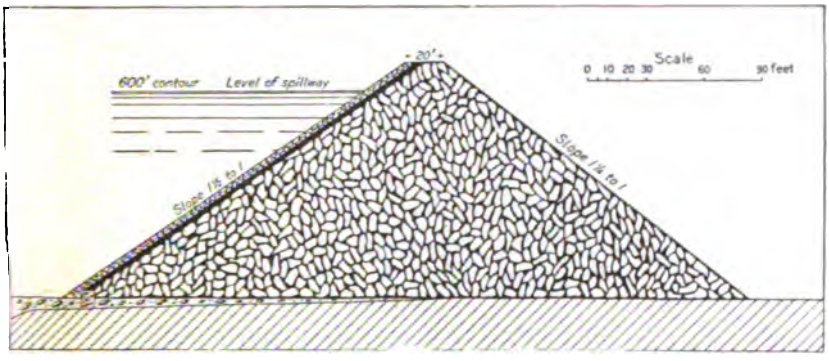


FIG. 16.—Maximum cross section of proposed Foster dam.

Concrete of this character, if laid on a 1½ horizontal to 1 vertical slope and protected, will certainly be permanent.

A cut-off wall of concrete is to be carried down to bed rock at the upstream toe of the dam. The asphaltic concrete is to join this wall, making the entire upstream slope of the dam water-tight.

The spillway channel is to be excavated in the Monterey shale at the south end of the dam. It is to be 200 feet wide on the bottom and will discharge approximately 29,000 second-feet with the water 12½ feet above the level of the crest. It is believed that this spillway will be permanent without a protective pavement. It should be deepened somewhat on the convex side to throw the current away from the south end of the dam.

The outlet from the reservoir is to be through a tower and tunnel near the south end of the dam, as shown in fig. 17. The tower is to be of concrete, is circular in plan, and will contain the valves for controlling the flow of the water from the reservoir. Four intake pipes 3 feet in diameter will be placed in this tower. The valves will be

the ordinary disks or covers, which will be lifted by a rod and geared wheels. The outlet gates from the tower are to be ordinary slide gates on steel rollers, with a clear opening of 2 by 4 feet. They will be operated by means of a rod and geared wheels, like the inlet valves. These outlet gates will normally remain open, but may be closed or partly closed in case the inlet valves should fail to close or break. They will also serve to regulate the pressure in the tower in case it is difficult to operate the inlet valves. As the water will be used solely for irrigation, all the inlet valves were placed near the bottom of the tower. If desirable, however, others can be placed at different levels at any time. The outlet tunnel is to be driven through the shale rock and lined with concrete to prevent erosion.

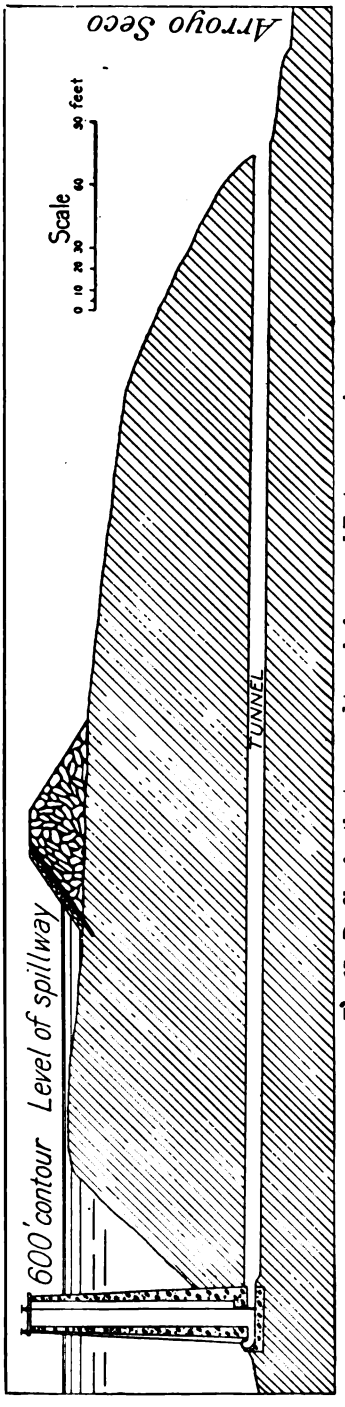


FIG. 17.—Profile of outlet tower and tunnel of proposed Foster reservoir.

The location of the spillway and dam is favorable for cheap and rapid handling of rock by means of two overhead cableways. The rock when blasted will doubtless shatter into sizes easily handled, probably small enough to be loaded into skips with a steam shovel.

The following is a summary of the estimated cost of the Foster dam and reservoir:

Estimated cost of Foster dam and reservoir.

Loose rock fill in dam	$141,920
Asphaltic concrete	51,240
Hand-laid rubble	14,738
Cut-off wall	7,173
Outlet tunnel	10,918
Outlet tower	9,630
Inlet valves	3,640
Outlet valves	3,900
Tower house and footbridge	500
Engineering and contingencies, 5 per cent	12,182
Reservoir and dam site	5,000
Total	260,841

The estimated cost of storage at this locality is $15.61 per acre-foot if the reservoir is filled once during the winter, or $7.80 per acre-foot if filled twice.

CURRIER RESERVOIR.

The proposed Currier reservoir is on Arroyo Seco, 11 miles south of Soledad in an air line and about 12 miles by the wagon road (Pl. II). The dam site lies in the NE. ¼ of sec. 19 and the NW. ¼ of sec. 20, T. 19 S., R. 6 E., M. D. M., and the reservoir site extends westward about 3 miles to the Foster dam site. A survey was made of this loca-

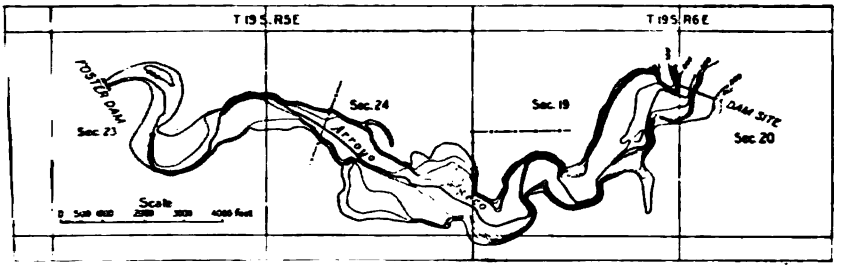


FIG. 18.—Currier reservoir site on Arroyo Seco.

tion by the writer in the summer of 1902. The topographic map of the reservoir is shown in fig. 18. The altitude of the stream at the dam site is 380 feet by aneroid observations. The survey was made for a dam which would raise the water 120 feet above the bed of the stream, or to the 500-foot contour. Such a dam will impound 24,657 acre-feet of water. The accompanying table shows the area and capacity of this reservoir at the various contours.

Area and capacity of proposed Currier reservoir, Arroyo Seco, California.

Contour.	Area.	Capacity.
	Acres.	*Acre-feet.*
380	0	0
390	7.8	39
400	30.2	229
410	79.5	777
420	112.8	1,738
430	139	2,997
440	186	4,622
450	231	6,707
460	282	9,272
470	329	12,327
480	375	15,847
490	441	19,927
500	505	24,657

Fig. 19 shows the capacity curve of the reservoir, by means of which the amount of water stored can be ascertained for any depth. The drainage area tributary to this reservoir is 189 square miles. The table on page 54 shows the estimated monthly discharge of Arroyo Seco at the Pettitt ranch, 1 mile below. The discharge here will be practically the same as at the gaging station. The reservoir is a portion of the terraced valley of Arroyo Seco. Pl. IX, *A*, shows the character of this valley looking upstream from below the dam site. The canyon has a width of about 340 feet at the bottom and

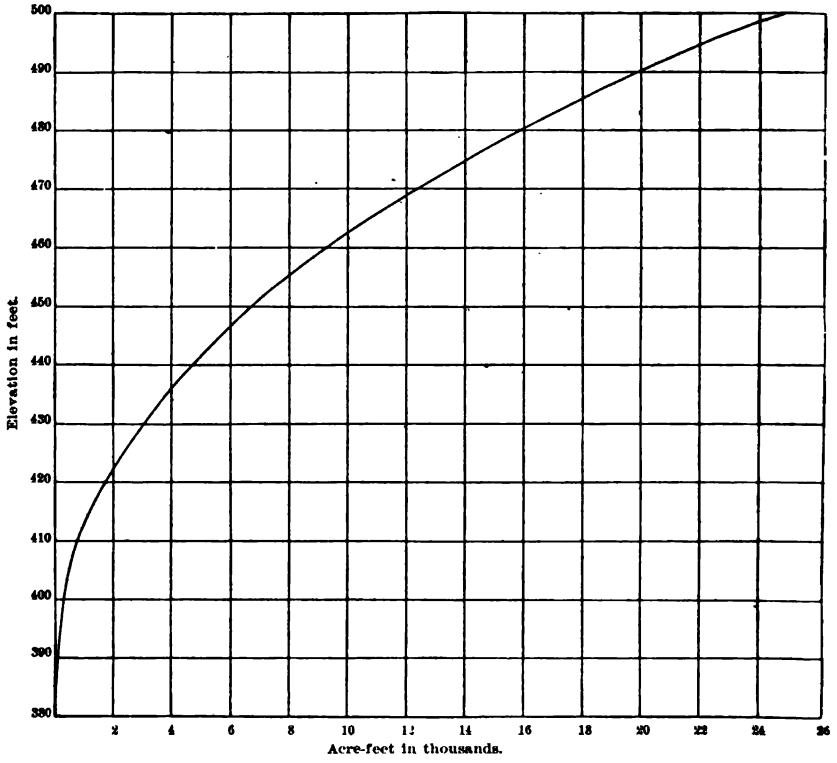
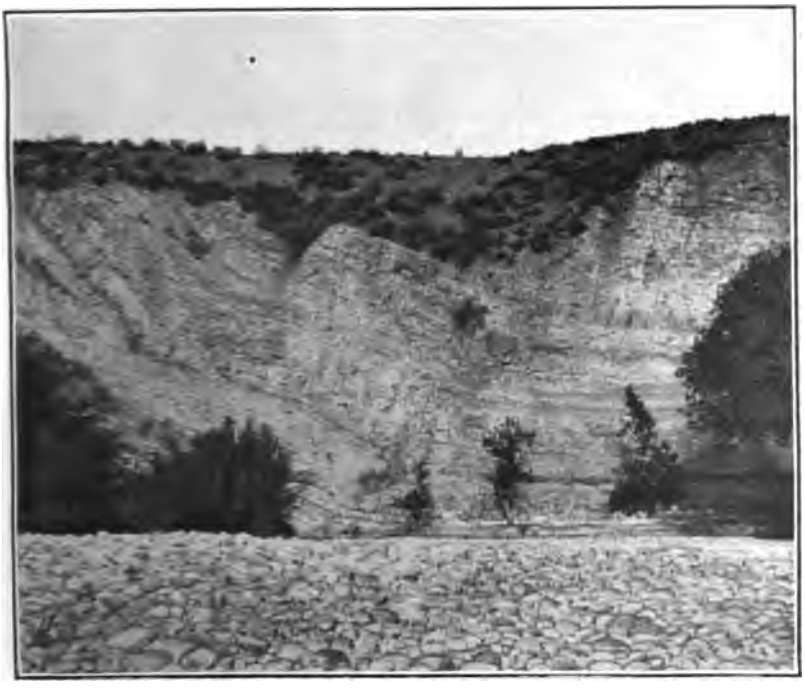


FIG. 19.—Capacity curve of proposed Currier reservoir, showing capacity in thousands of acre-feet, with the water surface at various levels.

of about 550 feet at the level of the first terrace, 125 feet above the stream. The rock at the dam site is the Monterey shale. This formation has been described on page 62. At this point the shales are evenly stratified in thin beds, which are sharply folded and somewhat faulted just below the axis of the dam. Pl. IX, *B*, shows this fold on the south side of the canyon and Pl. X, *A*, shows the nearly horizontal beds of shale in the north side of the dam site. The shale east of the axis of the fold strikes N. 70° W., and dips southwestward at an angle of from 40° to 45°. Rock is exposed in the sides

A. CURRIER DAM SITE ON ARROYO SECO, LOOKING UPSTREAM FROM BELOW THE DAM.



B. SOUTH ABUTMENT AT CURRIER DAM SITE.

of the canyon, but the bottom is covered with bowlders and sand. It seems probable that it is at least 20 to 25 feet to solid rock, but no borings have been made to test this locality.

PROPOSED CURRIER DAM.

The dam at this point as designed is to be of the rock-fill type. The location of the dam and spillway are shown in fig. 20. It is necessary either to excavate a spillway or to build a long wing wall to prevent the flood water from flowing around the end of the dam and over the cliff onto the lower slope. It was decided to excavate a spillway and use the rock in the dam. The dam will have a maximum height of 135 feet above the surface of the stream and will probably extend 20 to 25 feet below to bed rock, as shown in fig. 21. The upstream slope is to be 1½ horizontal to 1 vertical, and the downstream

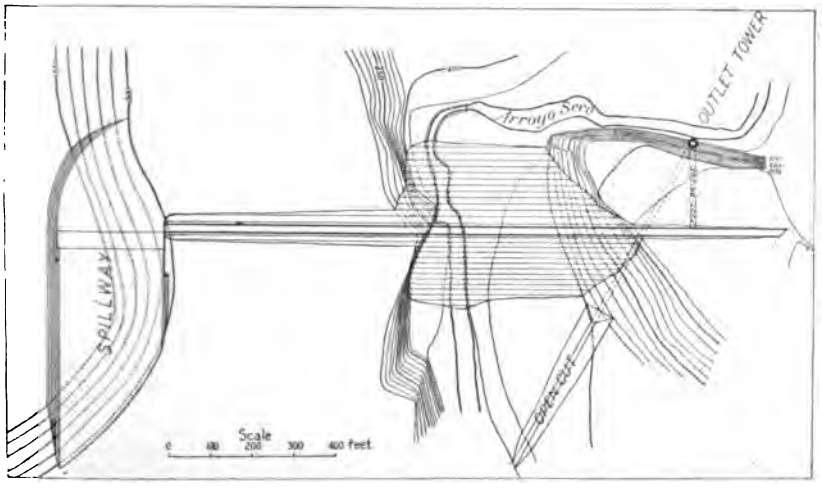


FIG. 20.—Currier dam site and spillway on Arroyo Seco.

slope 1¼ horizontal to 1 vertical. The top width is to be 20 feet. The extreme length of the dam will be 1,460 feet, but only the upper 20 feet of the dam will have this length, the balance being less than 500 feet long. The crest of the dam is to be 15 feet above the level of the spillway. In the future, if observations show that it is advisable, the level of the spillway can be easily raised. If raised 5 feet it would increase the capacity of the reservoir about 3,000 acre-feet. The dam is to be made water-tight by a 1½-foot layer of asphaltic concrete, laid on the upstream slope of the dam. This concrete is to be covered with 3½ feet of hand-laid rubble to protect it from the heat of the sun and weathering, and is to be of the same composition as that described on page 63 for use at the Foster dam. A cut-off wall of Portland-cement concrete is to be carried down to bed rock at the upstream toe of the dam. The asphaltic concrete is to join this wall, making the entire upstream slope of the dam water-tight.

The proposed spillway will have a breadth of 250 feet and ample capacity to handle any flood. It will discharge on a broad terrace several hundred feet from the edge of the cliff, and even if the shale does wear somewhat it is believed that the spillway will be permanent over the terrace. It is proposed to pave the spillway with asphal-

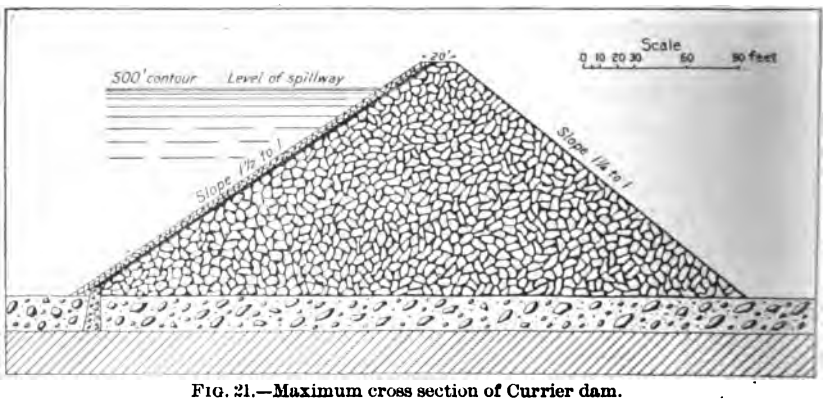


FIG. 21.—Maximum cross section of Currier dam.

tic concrete for 300 feet below the crest on account of the folded and fractured condition of the shale.

The outlet from the reservoir is to be through a tower and tunnel near the north end of the dam, as shown in fig. 22. The tower is to be of concrete, is circular in plan, and will contain the valves for controlling the flow from the reservoirs. Five intake pipes 3 feet in diameter will be placed in this tower. The valves will be the ordinary disks

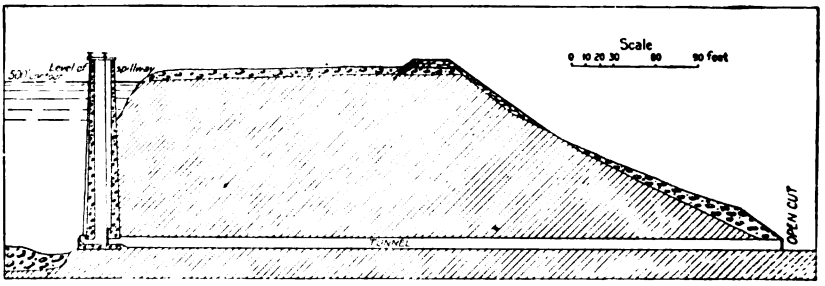


FIG 22.—Profile of outlet tower and tunnel of Currier reservoir.

or covers, which will be lifted by a rod and geared wheels. The outlet gates from the tower are to be ordinary slide gates on rollers, with a clear opening of 2 by 4 feet. They will be operated by means of a rod and geared wheels, like the inlet valves. These outlet valves will normally remain open, but may be closed or partly closed in case the inlet valves should fail to close or break. They will also serve to regulate the pressure in the tower in case it is difficult to operate the inlet valves. As the water will be used solely for irrigation, the

A. NORTH ABUTMENT AT CURRIER DAM SITE.

B. SUMMIT REGION OF MOUNT DIABLO RANGE AT HEAD OF GAVIOTA CREEK.

Shows scattered growth of timber and brush.

valves were placed near the bottom of the tower. If desirable, however, others can be placed at different levels at any time. The outlet tunnel is to be driven through shale rock and lined with concrete. As at the Foster site, the location of the spillway and dam is favorable for cheap and rapid handling of rock by means of two overhead cableways. The rock when blasted will doubtless shatter into sizes easily handled, probably into sizes small enough to be loaded into skips with a steam shovel.

The following is a summary of the estimated cost of the dam and reservoir:

Estimated cost of Currier dam and reservoir.

Loose rock fill in dam	$184,130
Asphaltic concrete	69,491
Hand-laid rubble	19,936
Cut-off wall	36,016
Wing wall north of spillway	7,704
Pavement on spillway	6,445
Outlet tunnel	11,895
Outlet tower	11,610
Inlet valves	4,550
Outlet valves	3,900
Tower house and footbridge	500
Engineering and contingencies, 5 per cent	17,809
Reservoir and dam site	7,500
Total	381,486

The estimated cost of storage at this locality is $15.47 per acre-foot if the reservoir is filled once during the winter, or $7.74 per acre-foot if filled twice, as described on page 85.

As the discharge of this stream is always sufficient to fill both the Foster and Currier reservoirs at least twice during the wet season, the outlet gates and tunnels were designed to discharge the contents of the reservoirs in thirty days. This amount of water can easily be handled in the valley below for winter irrigation. The impounded water can be released in the stream channel below the Currier dam and picked up by the existing and proposed canals at the mouth of the canyon, as shown on Pl. II.

SAN LORENZO CREEK.

The only stream of any importance entering the Salinas from the east, except Estrella Creek, is San Lorenzo Creek. (See Pls. I and II.) This stream leaves the Mount Diablo Range about 15 miles above its junction with the Salinas. Its easterly tributary, Lewis Creek, rises in the most elevated portion of the Mount Diablo Range, in Priest Valley, and flows northwestward in a deep valley in this range to within about 5 miles of its junction with Peachtree Creek, a stream that drains the southwestern slope of the long, narrow ridge between Priest Valley and the low-lying mesa lands to the southwest. This

ridge has an altitude ranging from 3,300 to 3,800 feet, and runs in a general northwest-southeast direction for many miles, being one of the main ridges of the Mount Diablo Range. These elevated ridges of this portion of the Mount Diablo Range are generally without forest covering, and often without even brush, bearing only scattered pine and oak trees. (See Pl. X, *B*.) Many large areas are covered only with a growth of grass or wild oats. In some of the valleys, as Priest Valley, there are large scattered oak trees. These ridges and elevated valleys, which embrace an area of 150 square miles, are in the region of greatest precipitation in the Mount Diablo Range, and the greater part of the run-off from San Lorenzo Creek in ordinary years is from this portion of the drainage basin, the mesa to the west yielding little or no run-off.

All of the streams that rise along the southwest slope of the Mount Diablo Range in Monterey County have eroded deep channels into the soft, sedimentary rocks of the mesa, the erosion being favored by the folded and broken condition of the strata in a narrow zone along the southwest slope of the Mount Diablo Range. The tendency of these streams—Peachtree, Gaviota, and Chalone creeks—to follow this folded zone is apparent on inspection of the drainage lines shown on Pl. I. The contour of the country is such that these streams receive but little drainage from the mesa which slopes southwestward to Salinas River. Peachtree Creek has recently been robbed of a portion of its drainage basin by Gaviota Creek, formerly one of the small mesa streams. This change of drainage was accomplished by the progressive erosion upstream of the canyon of Gaviota Creek until it finally cut through the dividing ridge and diverted the water from 40 square miles of the upper drainage area of Peachtree Creek. The comparative recency of this change is shown by the fact that Gaviota Creek is now rapidly cutting a deep gorge through the upper part of its course in the mesa, having already sunk its bed several hundred feet below the former bed of Peachtree Creek, and by the additional fact that the valley of Peachtree Creek below the diversion is fast filling with débris brought down by small lateral streams.

Below Lonoak San Lorenzo Creek has eroded a valley 500 to 800 feet deep, and has reached granite bed rock below the soft sedimentary rocks of the mesa (see Pl. XI, *A*).

From the scant information at hand it seems probable that the upper portion of this drainage area in the Mount Diablo Range receives an average annual precipitation of at least 20 inches, with a probable maximum for wet years of 40 inches, and a minimum of 10 inches for very dry years (see table on p. 43). During December, 1900, and from January 1 to April 30, 1901, and during 1902, a record was kept of the discharge of this stream at the Mathews ranch. From these observations the table below of discharge of San Lorenzo Creek has been prepared. The diagram shown in fig. 23 is a graphic illustration of the discharge of this stream during the season of 1901–2.

A. MATHEWS DAM SITE, LOOKING DOWNSTREAM, SHOWING GRANITE BED ROCK BELOW SOFT SHALES OF MESA REGION.

B. SPRECKELS SUGAR COMPANY'S PUMPING PLANT ON SOLEDAD RANCHO AT RANCH NO. 2.

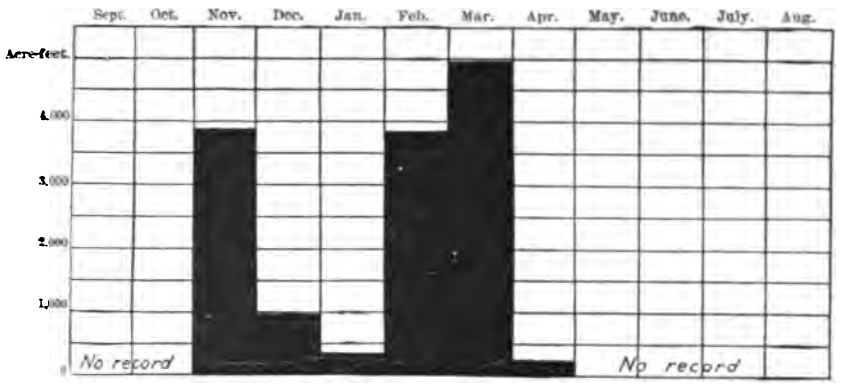


FIG. 23.—Discharge of San Lorenzo Creek at Mathews ranch from September 1, 1901, to August 30, 1902.

Estimated monthly discharge of San Lorenzo Creek at Mathews dam site, 5 miles east of Kings City.

[Drainage area, 235 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1900.						
December 16 to 31	13	9	9	286	0.04	0.02
1901.						
January	2,540	9	171	10,514	.73	.84
February	9,200	9	725	40,264	3.09	3.21
March	27	9	17	1,045	.07	.08
April	27	6	8	476	.03	.03
November	720	15	65	3,868	.28	.31
December	50	5	16	984	.07	.08
1902.						
January	15	5	6	369	.03	.03
February	795	5	69	3,832	.29	.30
March	875	5	81	4,980	.34	.39
April	5	5	4	238	.02	.02
November	200	15	85	5,057	.36	.40
December	15	5	7	430	.03	.03
1903.						
January	274	5	34	2,091	.14	.16
February	200	5	52	2,888	.22	.23
March	470	5	89	5,472	.38	.44
April	470	5	69	4,106	.29	.32
May	5	3	4	246	.02	.02

NOTE.—Gage heights and discharge measurement for 1901 are given in Water-Supply Paper No. 66, page 155; rating table on page 178, same paper.

During the summer of 1900 the lower part of this drainage basin was explored by Prof. Charles D. Marx. The Mathews site was discovered and a reconnaissance survey was made. During the summer of 1901 Mr. H. E. Green resurveyed the reservoir and dam site, designed the dam, and made estimates of the cost.

MATHEWS RESERVOIR.

The Mathews reservoir site is in the canyon of San Lorenzo Creek, about 6 miles northeast of Kings City. The dam site is in the NW. ¼ sec. 24, T. 19 S., R. 8 E., M. D. M., and the reservoir site extends eastward about 2 miles. The topographic map of the reservoir is shown in fig. 24. The altitude of the stream at the dam site is 440 feet. The survey was made for a dam that would raise the water 100 feet, to the 540-foot contour. Such a dam will impound 16,131 acre-feet of water.

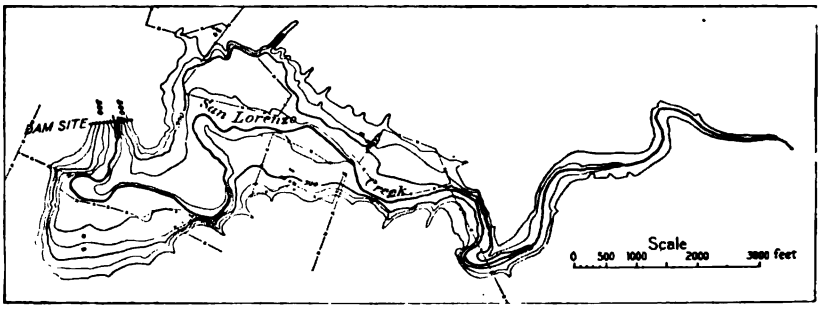


FIG. 24.—Mathews reservoir site on San Lorenzo Creek.

Area and capacity of proposed Mathews reservoir on San Lorenzo Creek.

Contour.	Area.	Capacity.
	Acres.	*Acre-feet.*
440	0.5	0
450	5.6	30.5
460	35.2	234.5
470	59.5	708
480	113	1,570.5
490	153.9	2,905
500	219	4,769.5
510	254.5	7,137
520	286.8	9,843.5
530	326	12,911.5
540	364	16,365.5

The accompanying table shows the area and capacity of this reservoir at the various contours. Fig. 25 shows the capacity curve of the

reservoir, by means of which the amount of water stored for any height can be ascertained. The area of the watershed tributary to this reservoir is 235 square miles. The table on page 71 shows the estimated monthly discharge from this stream at the dam site, and the table on page 43 shows the rainfall in the upper portion of the

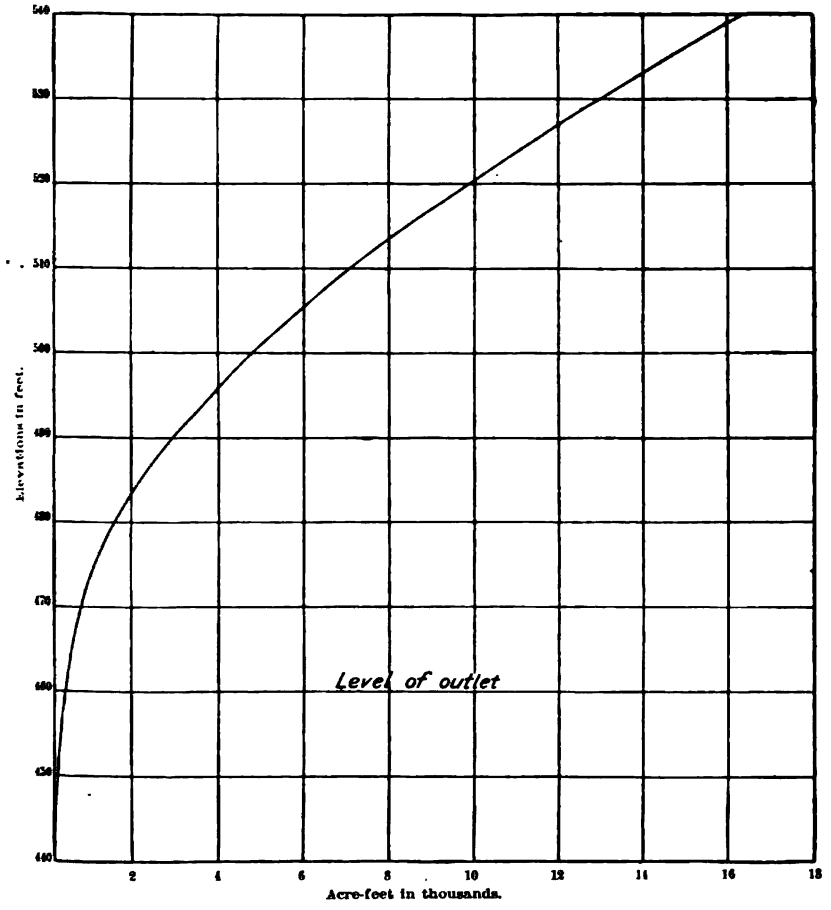


FIG. 25.—Capacity curve of proposed Mathews reservoir, showing capacity in thousands of acre-feet with the water surface at various levels.

drainage basin in Priest Valley. The reservoir is a portion of the canyon of San Lorenzo Creek, where it has cut down through the soft sedimentary formation of the mesa into the granite below. Pl. XI, *A*, shows the character of this canyon at the dam site. At the dam site the canyon has a width of 120 feet at the level of the stream and 590 feet at the top of the dam.

PROPOSED MATHEWS DAM.

The dam at this point as designed is to be an earth fill. The positions of the dam and spillway are shown in fig. 26. The dam will have a maximum height of 110 feet above the bed of the stream on

FIG. 26.—Mathews dam site and spillway on San Lorenzo Creek.

the axis of the dam, as shown in fig. 27. The upstream slope is to be 2½ horizontal to 1 vertical, and the downstream slope is to be 2 horizontal to 1 vertical. The top width will be 20 feet and the extreme length about 590 feet. The crest of the dam is to be 10 feet above the

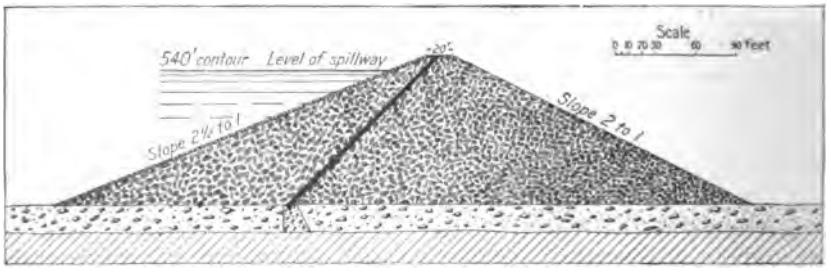


FIG. 27.—Maximum cross section of Mathews dam.

level of the spillway. The dam will be made water-tight by a layer of natural bituminous sandstone embedded in the dam, as shown in fig. 27. This bituminous rock can be obtained at the Mylar mine, 6½ miles upstream. It is of excellent quality and can be had in large

quantities. A cut-off wall of concrete is to be carried down to bed rock at the upstream toe of the bituminous layer. The depth to bed rock was estimated at 20 feet, but no test borings have been made here. The upstream slope of the dam will be protected from wave action by a 2-foot thick pavement of bowlders and coarse gravel.

The spillway will be excavated through a low saddle northeast of the dam site, as shown in fig. 26. It is to be 200 feet wide and 10 feet deep, lined with concrete for a short distance below the crest, if necessary. This spillway will discharge into a lateral canyon which reaches the main stream 500 feet below the dam site.

The outlet from the reservoir is to be through a tunnel in the granite bed rock and two 36-inch cast-iron pipes, as shown in fig. 28. The flow from the reservoir will be controlled by ordinary straight waygates or valves in this pipe, as shown in figs. 29 and 30. The flow will ordinarily be controlled by the lower gate; the other will be normally open, but can be closed in case of accident to the lower gates, or for repairs. The shaft is to give easy access to the gates. As the water will be used solely for irrigation, the outlet pipe was placed near the bottom of the reservoir, and no provision has been made for drawing off water at different levels.

Earth for the construction of a dam lies above the granite bed rock to the west of the dam. It can be delivered at the dam site either by a tramway or by overhead cableways.

The following is a summary of the estimated cost of the dam and reservoir:

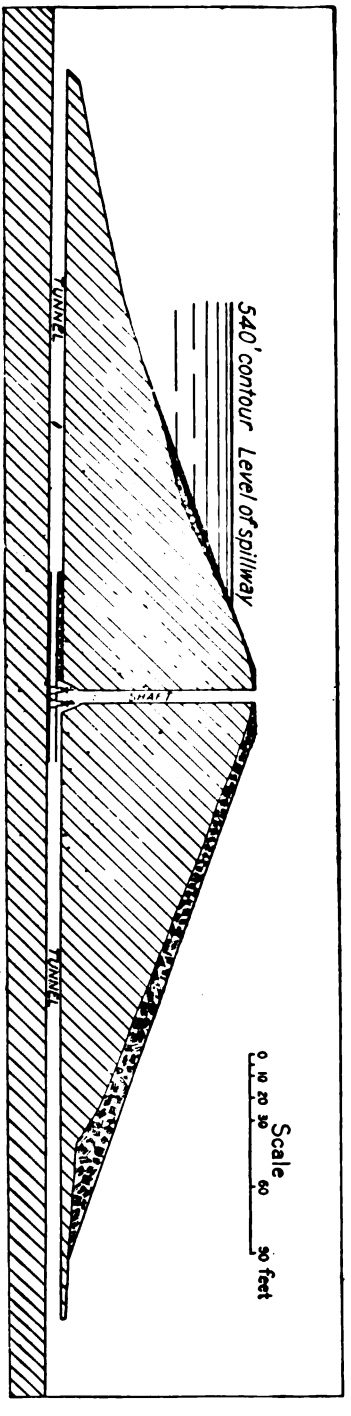


FIG. 28.—Profile of outlet tunnel and shaft of Mathews reservoir.

Estimated cost of Mathews dam and reservoir.

Earth embankment	$72,750.00
Bituminous rock core wall	44,240.00
Cut-off wall	12,112.00
Riprap	3,750.00
Outlet tunnel, shaft, and pipes	16,235.00
Overflow weir	4,800.00
Plant and material	25,000.00
Reservoir site and right of way	4,000.00
Engineering and contingencies	18,288.70
Total	201,175.70

Cost of storage per acre-foot, $12.29+.

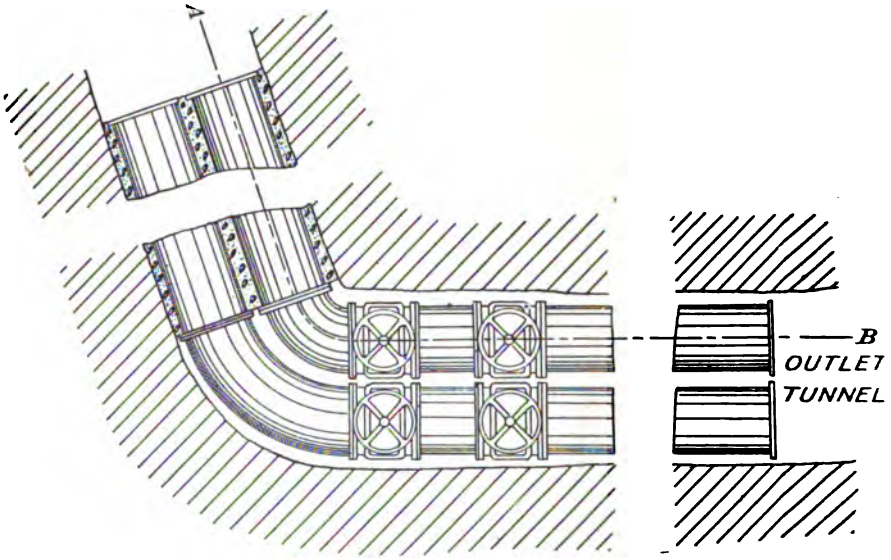


FIG. 29.—Plan of outlet gates of Mathews reservoir.

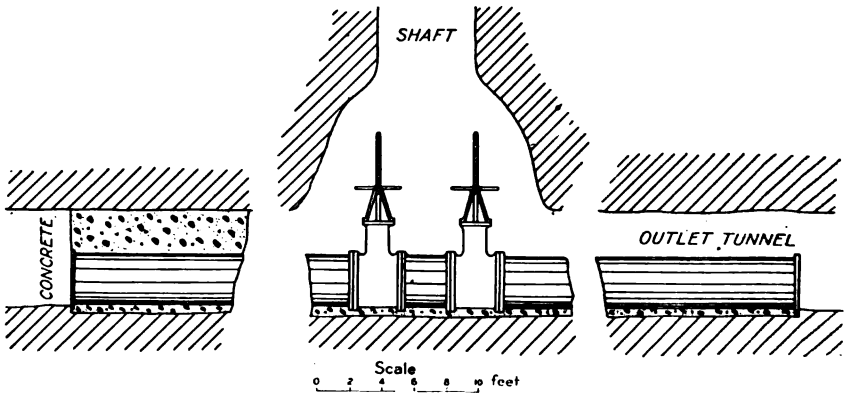


FIG. 30.—Vertical section on line A-B through outlet gates of proposed Mathews reservoir.

CHALONE CREEK.

Chalone Creek, which enters Salinas River near Metz (Pl. I), is a torrential stream flowing from the southeastern slopes of the Gabilan Range and Chalone Peak, draining an area of 128 square miles. It also has cut a deep canyon across the mesa lands. The discharge of this stream is large at times, as shown by the character of the channel. There are no reservoir sites in this watershed, and it is of small importance for irrigation purposes.

MESA STREAMS.

All the streams along the east side of Salinas River which rise in the mesa region, being without mountain drainage areas, are dry gulches during the dry season and flow but a few hours after storms in the wet season.

CARMEL RIVER.

Carmel River, although not a tributary of the Salinas, is one of the important streams of the region and merits a short description. It rises in a very rough mountain country on the northeastern slopes of the Santa Lucia Range and on the northwestern slopes of the Sierra de Salinas and flows northwestward, reaching Carmel Bay at the northwest extremity of the Santa Lucia Range (Pl. I). The valley is wide between the bounding hills and mountains for 10 miles from the coast, above which the stream flows in a canyon similar to that on the middle section of Arroyo Seco. Above Tularcitos Creek the stream flows in a canyon that reaches far back into the mountain region. Nothing very definite is known about the altitudes in this drainage area, but the average must be between 3,000 and 3,500 feet. The divide between Carmel River and Arroyo Seco is at an altitude of 2,500 feet where it is crossed by the trail. This portion of the Santa Lucia Range is covered with a very thick growth of brush and some areas are well forested by pines.

The Pacific Improvement Company diverts water from this stream by pipe system to reservoirs near Pacific Grove. The diversion dam is 15 miles from the mouth of the stream, at an elevation of 428 feet above sea level. The water is used for domestic supply at Pacific Grove and Monterey, and also for a small amount of irrigation.

The total area of the drainage basin of Carmel River is 275 square miles.

IRRIGATION.

CROPS IRRIGATED.

Only a comparatively small portion of the fertile lands of the Salinas Valley is now irrigated. The residents of this region have apparently not until lately realized the need of irrigation, but have placed entire dependence on the rainfall, while vast floods annually

run to waste. The cycle of dry years just past and the introduction of new crops have called public attention to the urgent need of supplementing the scant rainfall by irrigation. The present report is based on investigations undertaken to aid in the development of the water supply of this region. In early days all the upper Salinas Valley was given over to stock raising, the upper valley and mesa lands supporting vast herds. In the early seventies, when the railroad was built south to Soledad, a gradual change took place and wheat farming was begun in the valley. At the present time practically all the high valley land is used for raising wheat and barley, while the uplands or mesas, with the mountain slopes and small valleys, are still given over to stock raising. Grazing is and always will be an important industry. Pl. II shows the location of the irrigated and irrigable lands and the proposed storage reservoirs and canals in the Salinas Valley. It is estimated that there are 65,000 acres of irrigable land lying between the headworks of the Salinas canal and the ocean. Of this amount about 20,300 acres are irrigated by various ditches and pumping plants, leaving a balance of 44,700 acres of irrigable land to be provided for.

The principal crops irrigated are sugar beets, alfalfa, vegetables, and grain. Land on which sugar beets are raised is irrigated during the winter or early spring, usually but once, about 1½ acre-feet being run over the field. The amount of water used of course varies with difference in the porosity of soil, but 1½ acre-feet is a close average. This is in addition to the rainfall. Sugar beets are not irrigated after planting, but depend entirely upon one irrigation and the rainfall. Alfalfa is usually irrigated twice during the summer, about 1½ acre-feet being used for each irrigation, or a total of 3 acre-feet a year. Alfalfa is ordinarily cut four and sometimes five times a year. Land on which potatoes are raised is irrigated once, in early spring, before planting, about 1½ acre-feet being used. Grain is irrigated once, in April or May, about 1½ acre-feet being used.

The cost of irrigation from canals is about $1.50 per acre-foot, including profits to the canal companies. Irrigation by pumping is necessarily more expensive, the average cost, not including profits, being about $2.50 per irrigation, or $1.67 per acre-foot.

CANALS.

The following is a brief description of the irrigation works now in operation in the Salinas Valley.

The Salinas canal was constructed in 1896 and 1897. Water is diverted from Salinas River to this canal by means of a temporary dam near the north line of San Benito rancho, south of Kings City (Pl. II). The canal is 30 feet wide on the bottom, 40 feet wide on top, and was constructed to carry 5 feet of water. The grade is very light, 6 inches per mile, and the total length of the canal is about

9 miles. This canal was designed to divert water for winter and spring irrigation only, and now irrigates about 3,500 acres of land near Kings City, the principal crop being sugar beets and barley.

The San Lorenzo canal was constructed in 1896. Water is diverted from San Lorenzo Creek by means of a temporary dam of gravel at the mouth of the canyon, one-half mile below the Mathews dam site. (Pl. II). The canal is 20 feet wide on the bottom, 30 feet wide on top, and was designed to carry 5 feet of water. The grade is 5 feet per mile, and the total length of the canal about 8½ miles. This canal was designed for winter use only, and takes the flood water and winter flow from San Lorenzo Creek, which is practically a dry stream in the summer. The land irrigated is what is termed the San Lorenzo Creek bottom, and the amount irrigated each year has varied greatly, for since the construction of the canal there have been years of very scant rainfall. Perhaps 800 acres a year would cover the service of this canal.

Arroyo Seco canal No. 1 diverts flood water from the Arroyo Seco at the south line of lot 1 of the Arroyo Seco rancho (Pl. II). It was constructed in 1897. The canal is 25 feet wide on the bottom, 35 feet wide on top, and was designed to carry 5 feet of water. The grade is 5 feet per mile and the length is about 4 miles. This canal irrigates about 300 acres of land a year on the north half of the Arroyo Seco rancho.

Arroyo Seco canal No. 2 is owned by the Arroyo Seco Improvement Company. It was constructed in 1898. Water is diverted to this canal by a temporary dam of gravel at the mouth of the canyon of the Arroyo Seco, at the south line of the Arroyo Seco rancho (Pl. II). The canal is 17 feet wide on the bottom, 27 feet wide on top, and was constructed to carry 5 feet of water. The grade is 1 foot per mile and the total length about 4 miles. This canal irrigates about 4,000 acres of land each year on the south half of the Arroyo Seco rancho and on the Espinosa tract.

Arroyo Seco canal No. 3 was constructed by the Spreckels Sugar Company in 1901 and 1902. Water is diverted to this canal from the Arroyo Seco by a temporary dam about one-half mile below the head of canal No. 2 (Pl. II). The canal is 20 feet wide on the bottom, 38 feet wide on top, and was constructed to carry 4 feet of water. The grade is 5 feet per mile, and the length is 14 miles. It irrigates 2,000 acres of land on the Soledad rancho.[a]

The Gonzales Water Company's canal was constructed in 1899 at a cost of $18,375 for construction of canal and right of way. The main canal is 7½ miles long and diverts water from Salinas River by a wing dam of sand and brush about 4 miles south of the town of Gonzales (Pl. II). The river at this point flows throughout the year. The canal is 16 feet wide on the bottom, 32 feet wide on top,

[a] Information furnished by the Spreckels Sugar Company.

and has a grade of 1 foot per mile. About 2,700 acres of land are irrigated from this canal, principally grain land, but during the last summer somewhat over 500 acres have been irrigrated for alfalfa, beets, and beans. The land has proved to be well adapted to irrigation, being well drained and fertile. Good crops have been obtained when all else around was a failure.[a]

The Brandenstein ditch (now abandoned) diverted water from Salinas River in sec. 10, T. 23 S., R. 10 E., M. D. M., and irrigated land on the San Bernardino rancho, southeast of San Ardo. The survey was made for a canal 50 feet wide with a depth of 3 feet of water, and with a grade of 2 feet per mile. The main canal is about 6 miles long, and from 8 to 10 miles of lateral ditches have been constructed. No information is available as to the amount of water actually used or the exact location of the canals, so no attempt has been made to delineate them on the maps.[b]

PUMPING PLANTS.

Considerable water is pumped for irrigation at several points in the Salinas Valley. At the Spreckels Sugar Company's ranch No. 1, at the sugar factory, 4 miles south of Salinas (Pl. II), about 3,000 acres are irrigated, the waste water from the factory being utilized. The pump is an 18-inch Byron-Jackson centrifugal, directly connected with a General Electric 400-horsepower motor, driven from the factory generator. The average lift is 25 feet; the total average lift, including friction in pipe, 50 feet. The length of pipe is 4 miles; the diameters, 30, 32, and 34 inches. The general requirements of irrigation demand 20 inches in depth of water per annum. The pump can also take water directly from Salinas River for irrigation at any time. Water is spread by a system of permanent checks. The average size of the ditches is 3 feet on bottom, 12 feet on top, and 3½ feet deep.

The Spreckels Sugar Company also has a pumping plant on the Willoughby tract, near Spence station, on the right bank of Salinas River (Pl. II). The plant consists of one 20-inch Krugh centrifugal pump, belted to an Atlas compound engine. The pump takes water from Salinas River to irrigate 500 acres. The capacity of pump is 6,000 to 10,000 gallons per minute, with an average lift of 23 feet. The average area irrigated is 350 acres on alternate years. The average cost of pumping water is $2 per acre, or $1.20 per acre-foot, running daytime only. The consumption of fuel is one-third cord of wood per acre irrigated.

At the Spreckels Sugar Company's ranch No. 2—the Soledad rancho (Pl. II), opposite Soledad—about 500 acres are irrigated by water pumped from Salinas River. The pumping plant consists of one 20-inch Krugh centrifugal pump, belted to an 18 by 24 inch Atlas

[a] Information furnished by Dr. A. Gonzales, president Gonzales Water Company.
[b] Information furnished by Mr. Meyer Brandenstein.

SPRECKELS SUGAR COMPANY'S PUMPING PLANT NEAR KINGS CITY, AT RANCH NO. 3.

simple engine. The average lift is 22 feet, and the capacity of the pump is 6,000 to 10,000 gallons per minute. The consumption of fuel is at the rate of one-half cord of wood per acre irrigated. The cost of irrigation is $2.50 per acre, or $1.36 per acre-foot, running daytime only. The general requirements of irrigation demand 22 inches in depth in winter and spring. A general view of this pumping plant is shown in Pl. XI, *B*.

At the Spreckels Sugar Company's ranch No. 3 there is a pumping plant on Salinas River, near Kings City (Pl. II). It consists of three 20-inch Krugh centrifugal pumps, each belted to an 18 by 24 inch Atlas simple engine. Originally all the pumps took water from a sump fed from Salinas River and discharged into a 60-inch wood-stave pipe, 3,300 feet in length. Lately one pump has been connected with six wells, 20 inches in diameter and 70 feet deep, furnishing 6,000 gallons of water per minute. The capacity of each pump, when pumping from sump, is 8,000 to 10,000 gallons per minute. The lift is from 20 to 35 feet. The fuel used consists of willow and cottonwood. The total area susceptible of irrigation is 2,000 acres, partly controlled by the Salinas canal. About 400 acres of alfalfa are irrigated from the deep wells. Pl. XII is a general view of this plant, showing discharge pipes and 60-inch wood-stave pipe.[a]

The Salvation Army Colony tract on the Soledad rancho plant pumps water to a tract of land on the south side of Salinas River near Soledad (Pl. II). The source of supply is Salinas River, from which the water is diverted by an open cut.

The Soledad Land and Water Company has a pumping plant on the Soledad rancho, on the south side of Salinas River, where water is pumped for irrigating 800 acres of land near the old Soledad Mission. The soil is sandy loam, without hardpan, and requires about 1 acre-foot during the irrigating season, which extends from April 1 to November 1.

The pump is a centrifugal pump of 10,000 gallons' capacity per minute, with a lift of 20 feet. The plant is so arranged that water may be pumped to a height of 14, 20, 22, 27, or 31 feet, as required. The power is furnished by an 85-horsepower tandem, compound, Corliss engine, belted to the centrifugal pump. This plant cost $10,000 exclusive of the ditches. The actual running expense per hour, without taking into account the depreciation of the plant, is about as follows:

Engineer	$0.30
Superintendent	.25
Fuel	.25
Oil	.10
Incidentals	.10
Total	1.00

[a] Information furnished by the Spreckels Sugar Company.

The fuel used is willow and cottonwood, of rather poor quality, which is burned at the rate of about 1 cord every six hours. The cost of cutting and delivering wood on the company's land is $1.50 per cord. This plant is operated very economically and gives satisfaction.[a]

In the lower Salinas Valley there are many small pumping plants that pump from deep wells. The pumps are usually of the centrifugal type, operated by gasoline engines or occasionally by threshing engines, which are moved from place to place. Many of these wells will yield a second-foot, and it seems certain that there is a large supply of underground water in the vicinity of Salinas. These wells are used to irrigate fields of alfalfa, potatoes, and vegetables. The aggregate area thus irrigated was estimated to be approximately 2,600 acres. The underground water is discussed on pages 21–33.

EXTENSION OF IRRIGATION.

There are four sources of water supply available in this region for the extension of irrigation, viz: (1) Salinas River, (2) San Antonio River, (3) San Lorenzo Creek, and (4) Arroyo Seco. The location of these streams is shown on Pl. I, and the location of the most important reservoirs on Pl. II.

EXTENSION OF IRRIGATION FROM SALINAS RIVER.

The waters of Salinas River sink into the broad, sandy channel described on page 47, leaving but little surface flow in the dry season, when water is most needed. The winter floods are large, sweeping everything before them and converting the broad, sandy channel into a bed of shifting quicksand and silt. The estimated discharge of this stream from January 1 to July 31, 1901, is shown in the tables on page 48. The grade of this stream is very light, as noted on page 47, and the channel lies far below the general level of the valley. Any canals diverting water from this stream to the irrigable lands would of necessity have a very flat gradient and large cross section to deliver a sufficient amount of water. This adds greatly to the cost of construction. These features and the shifting channel of quicksand make it very difficult to divert the winter flow from the river. So serious are these obstacles that no large canals have been constructed to utilize the winter flow of the stream. Future investigation will, without doubt, show that it will be possible to divert Salinas River for winter irrigation, so as to conserve a portion of the water stored on the tributaries for summer use. There is an abundance of flood water for all the valley land that can be reached. Diversion from this stream will probably be most easily made by means of low, light weirs, such as are used on Kern River. They are inexpensive and in such a stream are very effective.

[a] Information by Mr. Ben Gould, Soledad, Cal.

Although the grade of Salinas River is very flat, the grades of its principal tributaries are much steeper, and a point is soon reached in ascending these streams where the flow may be diverted to the valley lands below.

EXTENSION OF IRRIGATION FROM SAN ANTONIO RIVER.

The most southern tributary of Salinas River from which it is practicable to divert water for the Salinas Valley is San Antonio River. The Pinkerton reservoir on this stream has been described on page 51.

By reference to the table on page 52 the cost of storage is seen to be excessive, but if a dam could be constructed to a height of 75 or 100 feet the storage capacity of the reservoir would be enormously increased. There is no probability of a shortage of water, as the run-off from San Antonio River is certainly large. There is not enough irrigable land along this stream for many miles below to justify the expense of a dam here. The only other land within reach of this reservoir is the Salinas Valley between Wunpost and Kings City. It is approximately 22 miles from the reservoir to Wunpost. The canal would be on steep sidehills for a large part of the way unless it could be run through Hames Valley, in which case the distance would be considerably shortened. It would be necessary to cross Salinas River with an expensive flume to reach the northeast side of the valley.

For these reasons no further investigations were made of this project. It is possible, however, that in the future this site may become a valuable adjunct to the irrigation systems in the Salinas Valley. A cheaper dam, of the rock-fill type, could be substituted for the masonry structure, and this would greatly reduce the cost of storage.

EXTENSION OF IRRIGATION FROM SAN LORENZO CREEK.

The proposed Mathews reservoir and dam have been described on page 72. From December 16, 1900, to April 30, 1901, inclusive, the estimated discharge of this stream was 52,585 acre-feet (see table on p. 71), or nearly three and one-fourth times the capacity of the reservoir. From November 1, 1901, to April 30, 1902, inclusive, the estimated discharge of this stream was but 14,271 acre-feet (see table on p. 71), or but nine-tenths the capacity of the reservoir. As the rainfall this season was undoubtedly somewhat below the normal, it is probable that the reservoir will be filled in a season of average rainfall, while much water would go to waste in a wet season. From November 1, 1902, to May 30, 1903, inclusive, the estimated discharge of this stream was 20,290 acre-feet (see table on p. 71), or one and one-fourth times the capacity of the reservoir. Residents along the stream are authority for the statement that in some very dry seasons the stream flows little or none in the lower portion. During the summer the water is very alkaline. A sample collected one-fourth

mile above the head-gate of the San Lorenzo canal in the summer of 1901 and analyzed shows that when the stream is at the summer stage, discharging about 1 second-foot, the water may be expected to contain about 480 parts of solids in 100,000 parts of water. The complete analysis is shown below, expressed in parts per 100,000, the elements being calculated as compounds.

Analysis of water from San Lorenzo Creek during summer flow.[a]

	Parts in 100,000.
Calcium sulphate ($CaSO_4$)	80.15
Magnesium sulphate ($MgSO_4$)	57.40
Sodium sulphate (Na_2SO_4)	175.22
Potassium chloride (KCl)	10.68
Sodium chloride (NaCl)	119.35
Sodium bicarbonate ($NaHCO_3$)	34.11
Sodium carbonate (Na_2CO_3)	4.19
Total	481.10

Samples of water were also collected from the first rises at the beginning of the wet season, No. 1 when the stream had risen 1 foot and No. 2 when it had risen approximately 2 feet. The proportion of solids is still seen to be very large.

Analyses of water from San Lorenzo Creek at beginning of wet season.[b]

	No. 1.	No. 2.
	Parts in 100,000.	*Parts in 100,000.*
Calcium sulphate ($CaSO_4$)	53.3	114.9
Magnesium sulphate ($MgSO_4$)	74.1	85.8
Sodium sulphate (Na_2SO_4)	37.9	40.4
Potassium chloride (KCl)	15.6	22.5
Sodium chloride (NaCl)	40.1	17.1
Sodium bicarbonate ($NaHCO_3$)	43.7	74.9
Total	264.7	355.6

No samples have been collected from this stream after prolonged floods, but no doubt the amount of alkali is much less in time of flood. Provision should be made for drawing off the summer flow and the first flood waters before filling the reservoir. The total mineral content of the low-water flow and first floods is so large as to produce an appreciable and ultimately deleterious deposit in irrigated soil. The stream carries a large amount of silt in floods, but no determinations have been made of the percentages or the time it will take to fill the reservoir.

As no run-off records were kept on this stream prior to 1900, there are few data on which to base a reliable estimate of the discharge. It

[a] U. S. Department of Agriculture, Bureau of Soils. Atherton Seidell, analyst.
[b] U. S. Department of Agriculture, Bureau of Soils. Bailey E. Brown, analyst.

seems probable, however, that dry seasons may occur once in from six to ten years, when little water will be stored, and a supply must be held over from the preceding season. It is estimated that 4,000 acre-feet should be held over to maintain crops that require a continuous supply of water, as alfalfa, etc. By this estimate the effective capacity of the reservoir is reduced from about 16,000 acre-feet to about 12,000 acre-feet in ordinary seasons. Assuming the duty of water at 2 acre-feet per season, this reservoir should provide water for 6,000 acres in ordinary seasons with a hold-over supply of 4,000 acre-feet.

During seasons of more than average rainfall if the water for winter irrigation is drawn from the reservoir it will serve as a regulator of the flood discharge, and more land can be irrigated if crops requiring winter irrigation only are raised.

Canals are already constructed from the mouth of the canyon to and beyond Kings City, but it will be necessary to extend the system if the reservoir is built; however, no expensive works will be necessary.

EXTENSION OF IRRIGATION FROM ARROYO SECO.

The proposed Foster and Currier reservoirs have been described on pages 59 and 65. The table on page 54 shows that the total estimated discharge of Arroyo Seco from January 1 to August 31, 1901, was 144,602 acre-feet, approximately 3.5 times the combined capacity of both reservoirs. One large flood occurred in November, 1900, of which there is no record, as the gaging station had not yet been established, but it is certain that a great amount of water was discharged.

The table on page 54 shows that from September 1, 1901, to August 31, 1902, the total estimated discharge of this stream was 100,947 acre-feet, or approximately 2.4 times the capacity of both reservoirs. The rainfall throughout the Salinas Valley was somewhat below the average this season. The table on page 54 shows that from September 1, 1902, to August 31, 1903, the total estimated discharge of the stream was 104,024 acre-feet, 2½ times the combined capacity of both reservoirs. No other records of the discharge of this stream have been kept, and no estimate can be made of the discharge for a very dry season, such as that of 1897–98, but from the testimony of residents in this region it seems probable that both reservoirs can be filled once even in seasons of very scant rainfall, and no hold-over storage will be necessary.

In seasons of ordinary rainfall the discharge of the stream is sufficient to fill both reservoirs twice and in wet years three or more times. As noted on page 78, it is desirable to irrigate for certain crops during the winter and early spring, say from January 1 to March 31. This ordinarily is the season of greatest discharge from the Arroyo Seco.

It seems probable that by proper management both reservoirs can be filled and emptied and filled again in this period in seasons of ordinary rainfall and filled and emptied twice and filled again in wet sea-

sons. As noted above, they can probably be filled once even in the driest years. The combined capacity of the reservoirs is 41,367 acre-feet. By this estimate it is seen that it will be possible to start at the beginning of the dry season with full reservoirs, which, with the duty of water at 2 acre-feet, will provide irrigation for about 20,000 acres in very dry seasons. This is sufficient to carry through a dry year such crops as alfalfa, as well as orchards, etc. In seasons of ordinary rainfall the reservoirs should provide water for irrigating 40,000 acres, and in wet seasons 60,000 acres or more. It is believed that this estimate is conservative.

The reservoirs are thus seen to serve as regulators of the flood discharge as well as storage reservoirs.

To carry this water to the lands below it will be necessary to construct a canal approximately as located on Pl. II. This canal can be so located as to divert flood water from Salinas River between Kings City and Soledad, and the canals from the San Lorenzo and Arroyo Seco could discharge into it during the latter part of the season, supplementing the winter flow, if necessary. It is believed that such a canal, in connection with the reservoirs, will supply water to all the best irrigable land from Soledad to the Bay of Monterey. No surveys for such a canal and no estimates of its cost have been made by the United States Geological Survey.

The stream carries but a small percentage of silt, and the water is very pure, in every way suited for irrigation and domestic use.

The following table of analysis shows the character of this water at the gaging station the first of the season after a slight rise:

Analysis of water from Arroyo Seco.[a]

	Parts in 100,000.
Calcium sulphate ($CaSO_4$)	4.0
Magnesium sulphate ($MgSO_4$)	2.9
Potassium chloride (KCl)	1.0
Sodium bicarbonate ($NaHCO_3$)	13.8
Potassium bicarbonate ($KHCO_3$)	.8
Total	22.5

Other analyses show that the flood water contains slightly less mineral matter.

EXTENSION OF IRRIGATION BY PUMPING WATER.

If the above-described reservoirs and canals are constructed it is probable that pumping water for irrigation will decrease rather than increase in the lower Salinas Valley. The high price of fuel makes pumping expensive and in the future it will undoubtedly be limited to procuring water for domestic supplies and for the irrigation of small garden patches. The underground water conditions of the

[a] Calculated as compounds. U. S. Department of Agriculture, Bureau of Soils. Bailey E. Brown, analyst.

lower Salinas Valley have been described on page 21. In the upper Salinas Valley and the mesa regions it is not improbable that sufficient water for stock-raising purposes could be reached by deep wells. Deep-well pumps driven by powerful windmills would raise the water to the surface and make available range lands now unused. For a detailed description of pumps and windmills used for such purposes the reader is referred to Water-Supply and Irrigation Papers Nos. 1, 8, 14, 29, 41, and 42 of the United States Geological Survey, and for a description of the methods of irrigation from wells to Water-Supply and Irrigation Paper No. 20.

SUMMARY.

The important results of the hydrographic investigations are summarized in the following table:

Summary of results of investigations.

Improvement.	Capacity in acre-feet.	Cost of installation.		Annual charge for interest, taxes, and maintenance.	Annual charges per acre-foot stored.
		Total.	Per acre-foot stored.		
Foster reservoir, 125-foot dam:					
In dry years	16,710	$260,841.00	$15.61	$41,237.00	$1.28
In ordinary years	33,420	260,841.00	7.80	41,237.00	.64
In wet years	50,130	260,841.00	5.20	41,237.00	.43
Currier reservoir, 135-foot dam:					
In dry years	24,657	381,486.00	15.47	29,441.00	1.19
In ordinary years	49,300	381,486.00	7.74	29,441.00	.60
In wet years	73,971	381,486.00	5.16	29,441.00	.40
Mathews reservoir, 110-foot dam:					
In dry years	4,000	201,175.70	50.29	17,180.00	4.30
In ordinary years	12,000	201,175.70	16.76	17,180.00	1.43
In wet years	16,131	201,175.70	12.29	17,180.00	1.07

In the above estimate interest was calculated at 6 per cent and taxes at 0.8 of 1 per cent on the total cost of the dams and reservoirs. In each case an additional charge of $3,500 per annum was made for maintenance and supervision.

The last column of the foregoing table gives the cost per acre-foot per annum of water stored at the above-mentioned reservoirs. Data are not available to estimate the loss by seepage in canals and by evaporation, but in estimating the amount of land that can be irrigated a liberal allowance has been made of the duty of water. It is assumed that for each 1.5 acre-feet of water put on the land 0.5 of an acre-foot will be lost in the canals. About 1.5 acre-feet per annum is the average amount of water actually used on the land. The Leigh reservoir and Pinkerton reservoir are not of special value at present, but may be valuable adjuncts to the water supply of the Salinas Valley in the future.

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DEPARTMENT OF THE INTERIOR

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

GEOLOGY AND WATER RESOURCES

OF PART OF THE

LOWER JAMES RIVER VALLEY, SOUTH DAKOTA

BY

J. E. TODD and C. M. HALL

WASHINGTON
GOVERNMENT PRINTING OFFICE
1904

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
HYDROGRAPHIC BRANCH,
Washington, D. C., July 21, 1903.

SIR: I have the honor to transmit herewith, for publication in the Water-Supply and Irrigation series, a paper on the "Geology and Water Resources of a Portion of the Lower James River Valley, South Dakota," prepared by Messrs. J. E. Todd and the late C. M. Hall, acting under the direction of Mr. N. H. Darton.

The region is a portion of the territory of which Mr. Darton made a reconnaissance several years ago. Professors Todd and Hall have obtained a large amount of new data, and this is presented in maps and diagrams which, it is believed, will prove of interest and value to many persons.

Very respectfully,

F. H. NEWELL,
Chief Engineer.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

GEOLOGY AND WATER RESOURCES OF PART OF THE LOWER JAMES RIVER VALLEY, SOUTH DAKOTA.

By J. E. Todd and C. M. Hall.

LOCATION AND IMPORTANCE OF THE REGION.

The region treated in this paper includes the Alexandria, Mitchell, Huron, and De Smet quadrangles. It is bounded by parallels 43° 30′ and 44° 30′ north latitude and meridians 97° 30′ and 98° 30′ west longitude. It embraces the whole of Davison, Hanson, and Sanborn counties, the most of Beadle and Miner, and portions of Kingsbury, Jerauld, Aurora, and McCook counties, S. Dak. It is traversed from north to south by James River Valley, along which it extends for about 75 miles.

The peculiar geologic interest in this region arises from the presence in its southern portion of a low buried mountain range, carved out of pre-Cambrian rocks, over which Cretaceous strata were laid down until the whole became a plain. The way in which this was accomplished has been revealed clearly by the numerous borings made for artesian wells. The region also exhibits representative portions of the Gary and Antelope moraines of the Wisconsin age of the Glacial epoch.

The economic value of the investigation here presented is mainly connected with the underground water resources, for the greater part of the region lies in the artesian basin. The number of water-bearing strata, the pressure of the water, and various other questions connected with the development of this resource have been studied with care.

TOPOGRAPHY.

In general the surface is very even, none of it being too rough or too steep for plowing except in the immediate vicinity of the larger streams and in a few ravines which are cut in its steeper slopes. Much of the region is nearly level, but as a whole it presents considerable range of altitude. James River crosses it near the center in an almost direct south-southeast direction. Its northeast corner lies on the western rim of the Coteau des Prairies, at an altitude of over 1,800 feet above sea level, and its southwest corner rises on the opposite side of the valley to an altitude of nearly 1,700 feet, while the general level of the wide valley between averages about 1,300 feet.

The highest point is about the middle of the east line of sec. 12, T. 111 N., R. 57 W., where an elevation of about 1,850 feet above sea level is reached. Near the southwest corner the highest point is at an

altitude of 1,690 feet, on the middle of the south line of sec. 9, T. 107 N., R. 63 W. The lowest points are along James River, the surface of which has an altitude of about 1,200 feet in the southern and 1,230 feet in the northern portion of the region.

GEOGRAPHY.

The whole surface was originally prairie with the exception of a few small groves along James River. One of these which attracted much attention in the early settlement of the country lies in a bend of James River near Forestburg. The whole surface is underlain by a clayey subsoil with the exception of some sandy areas in the vicinities of Letcher, Forestburg, and Huron. North and northwest of Forestburg the sand is so extensive that sand hills 15 to 30 feet in height have been formed. One of these opposite the mouth of Redstone Creek is known as Belchers Mound.

The area is drained mainly by James River and its tributaries. None of the tributaries contain flowing water throughout the year except Sand Creek for 7 or 8 miles above its mouth and Pearl Creek for half that distance. James River flows in a trough about half a mile wide and from 50 to 80 feet deep. It is a sluggish stream, having a width of 80 to 100 feet in ordinary stages and a depth of 3 to 10 feet. Its bottom land is fertile, but much of it is subject to occasional overflow.

The arrangement of the streams has been mainly determined by the movements of the ice sheet during the Glacial epoch, as will be explained in a subsequent section.

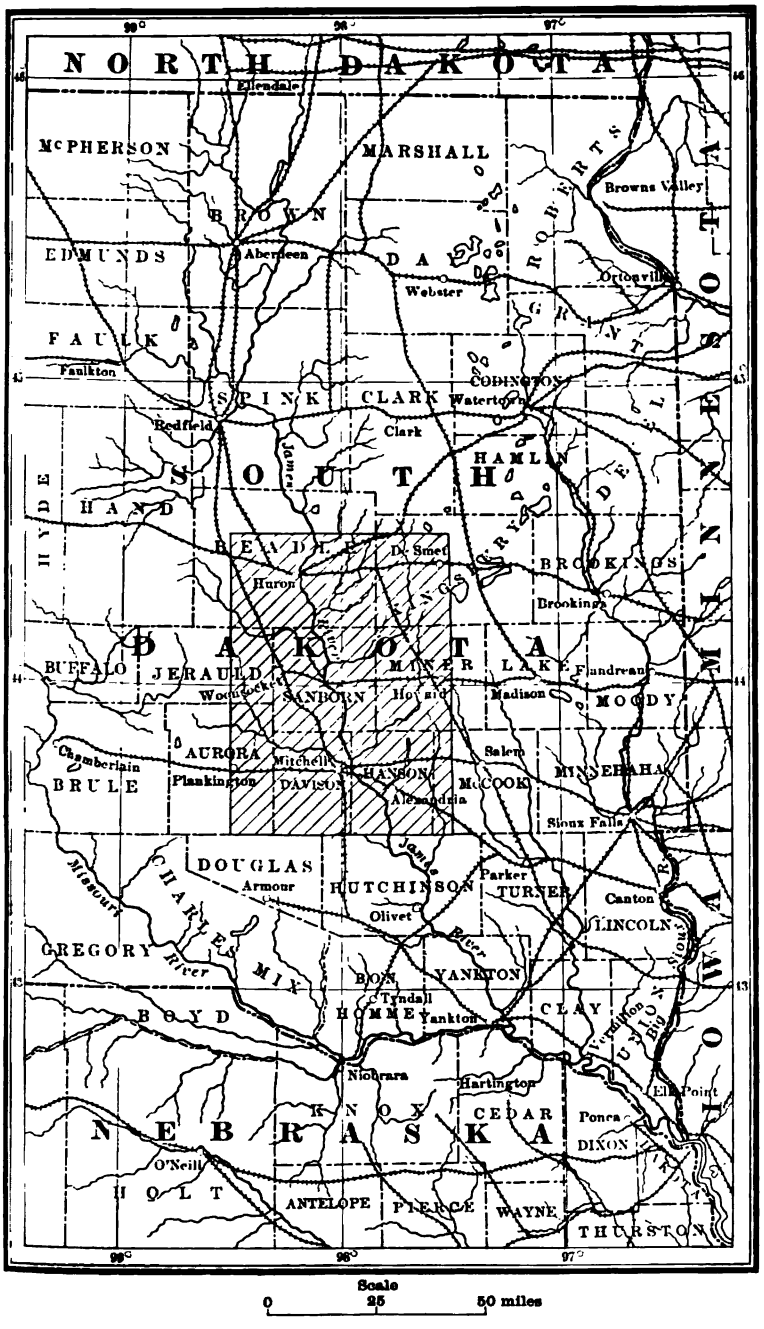
GEOLOGY.

The geology of this area is simple. Nearly the whole surface is covered with the glacial clays and stream deposits of the Quaternary. The rocks lie nearly horizontal and there are no traces of recent igneous action, exposures of older rocks occurring only in the southern quarter.

The following rocks occur in this area: The Algonkian, the Dakota, Benton, Niobrara, and Pierre formations, and the glacial deposits of the Quaternary. (See Pl. III.)

ALGONKIAN.

The crystalline rocks are represented by two formations, which were possibly formed at widely separated times. The first is the granite, which is nowhere exposed but has been struck in drilling wells at several points, and the other is the Sioux quartzite, which outcrops east of Mitchell and is often found in drilling. There are two theories concerning the relation of the granite to the quartzite. According to the first it is a much older formation, corresponding in age to the granite extensively exposed along Minnesota River below Bigstone Lake, which is believed to antedate the schists and quartzites of Minnesota and the Black Hills. According to the other view this granite



INDEX MAP OF EASTERN SOUTH DAKOTA, SHOWING AREA UNDER CONSIDERATION.

is an eruptive rock, which has been intruded into the quartzite in the form of dikes or sheets. If this is the case, it corresponds to an igneous rock forming a dike in the quartzite north of Corson, S. Dak. In favor of the latter view may be urged the evident unevenness of its surface. For example, it has been struck at the depth of 500 feet, or 870 feet above the sea, in sec. 25, T. 103 N., R. 61 W., while at Mitchell, less than 5 miles away, at a depth of 710 feet, or 590 feet above the sea, the quartzite, which according to the first view must lie above it, had not been penetrated; or, in other words, the granite had not been reached.

Besides the instance already mentioned, the granite has been struck in two or three wells about 5 miles north of Farmer, and a little beyond the north line of this area near Hitchcock. In the latter case it seemed to be overlain by several feet of quartzite.

The granite from the Budlong and Motley wells north of Hitchcock and from wells north of Farmer, in Hanson County, is a fine-grained, light-gray rock abounding in transparent feldspar, while that from the wells southwest of Mitchell is darker and coarser.

From wells at Huron and near Esmond it appears that below the water-bearing rock there are several feet of secondary deposits from granite, such as arkose, impure kaolin strata, and the like, then weathered granite before the sound rock is reached.

The deepest well at Huron, city well No. 4, opposite the college, according to the report of Mr. F. H. Holton, who drilled it and submitted specimens of the lower strata, shows the following record:

Record of deep well at Huron, S. Dak.

	Thickness.	Depth.
	Feet.	*Feet.*
First strong flow, probably top of Dakota		740
Unreported, but doubtless mostly shale	124	864
Hard rock	6	870
Sandstone and water flow	3	873
Unreported, but mostly shale	60	933
Sandstone and water flow	3	936
Black shale	125	1,061
Pebbly sand and water flow	9	1,070
Coarse sand with rounded grains about the size of No. 6 shot, largely reddish-brown concretionary grains with some white quartz-like material struck at corresponding levels in the Wilcox well and city well No. 3. The brown grains are apparently siderite, or carbonate of iron	30	1,100
Gray sandy rock with green and brown specks and whitish opaque grains like weathered feldspar (arkose), and lumps of coarse red granite	36	1,136
Soft, fine-grained, kaolin-like clay	2	1,138
Hard rock, probably granite	1	1,139

The Sioux quartzite, which is exposed in the southeastern of the area under consideration, is commonly a dense and fir fied rock. It is usually fine grained and thick bedded, bu cases a few feet of it contains numerous pebbles, while quently the rock is so thin bedded as to be worthless for purposes. Moreover, at some points it is so imperfectly con that it may be dug with a pick. A case of this sort occurs s of Bridgewater, in the valley of Wolf Creek. This soft ro ever, is very limited in extent, the normal hard quartzite o within a few feet of it. Associated with the quartzite and in fied with it are occasional thin beds of a red, hardened cla "pipestone." When this has been exposed to the weather it chalk-white. Examples of the latter condition are found at Creek locality.

The peculiar interest of the quartzite in connection with the under consideration is the fact that it is the bed rock ov area. The water-bearing strata rest upon it and its presenc the lowest horizon at which a flow may be obtained. Henc importance to be able to recognize it and to have a general edge of its depth below the surface.

In drilling, the quartzite may be distinguished from pyrite is of about the same hardness, by its thickness, the latter bein more than a few inches thick. Greater thickness also serve tinguish it from some of the hard layers of the Dakota for which are rarely over 4 or 5 feet thick. Moreover, the Dako are usually cemented by carbonate of lime, which effervesc acids, or by iron oxide or carbonate of iron, which is of a dark color, while the quartzite is uniformly of a light pinkish shade. ever, in some cases a compound microscope is necessary to de difference. When so examined quartzite is recognized by clu sand grains cemented together so firmly by glassy silica that the original grains divide along the fractures as easily as they s where cemented. Similar examination suffices to show the p of granite, which, as has already been stated, has been struck eral points in this area.

BED-ROCK CONTOUR.

In general the quartzite underlies the whole area under con tion and has the configuration shown by the contour lines in P The most conspicuous feature of the bed-rock surface is a high having a breadth of about 12 miles, which enters the east side area, affording surface exposures in the vicinity of Spence Bridgewater. Its outcrop area narrows rapidly toward the w that the westernmost appearance of the rock on the surface Enemy Creek, in the western part of T. 102 N., R. 59 W. triangle between the limits indicated it lies next underneat drift and shows frequently in the bottom of the valleys of the

AKUTA

streams. It has an altitude of 1,380 feet at Spencer and about the same near Bridgewater. On Enemy Creek it rises to only about 1,200 feet. Its most extensive exposure is in the bottom of James River Valley at Rockport. It has been traced farther west at lower levels in a zone of considerable width, and about a mile southwest of Mitchell has been struck at a depth of nearly 258 feet, or about 1,087 feet above sea level. At Plankinton it was found at a depth of 800 feet, or 700 feet above sea level.

This ridge is not a smooth anticline, but is composed of strata dipping in different directions at from 2° to 5°. It seems to be much eroded in places, where portions of the original surface are left as peaks between the valleys. Differences of elevation of 300 feet or more have been found within the distance of a mile. The northern slopes of this ridge seem to be generally rougher than the southern, and the valleys are correspondingly deeper and narrower. Toward the north the general surface of bed rock drops 600 feet in 8 or 10 miles from the last-observed outcrops. Then by a gentler slope it reaches a depth of 1,000 feet, or 300 feet above sea level, and continues northward till it rises slightly near the northwestern part of this area. Southward the slope is more gentle and a depth of 600 feet is not reached within 20 miles of the last outcrop, which carries the observation much beyond the limit of this area. These general facts are derived from well records, as the wells strike the quartzite very frequently in the vicinity of the ridge, but rarely far from it. The various sections and maps published in this paper exhibit such details as are available concerning the relations and distribution of the quartzite.

PALEOZOIC GAP.

In this region there are no traces, even in the numerous borings which have reached the Algonkian, of any Paleozoic formations or of any Triassic or Jurassic strata. The nearest occurrences of Paleozoic rocks that have been discovered are in the borings at Ponca, Nebr., and Sioux City, Iowa. During Paleozoic times, when great masses of sediments were being deposited in many other regions, this area was probably an elevated land surface. It is possible that soils and vegetation may have extended over it, and possibly some thin deposits which were removed by the advance of the sea during Cretaceous and earlier times. At any rate no traces of such deposits have been found in this vicinity on the surface of the quartzite. Since several hundred feet of strata of marine origin representing all the ages of Paleozoic times are found in the Black Hills, the shore of the Paleozoic sea probably extended across South Dakota from north to south, west of the present course of Missouri River. Moreover, the deep erosion of the intensely hard Sioux quartzite indicates that it was subject to erosion for a great length of time.

CRETACEOUS.

Resting upon the Sioux quartzite are sands, sandstones, chalk, and clays belonging to the Cretaceous series, comprising the Dakota sandstone, Benton formation, Niobrara chalk, and Pierre shale. As these are intimately connected with the sinking of artesian wells, it is desirable to describe them at some length, although some of them are exposed only in very limited areas over a surface of perhaps 150 square miles, mainly west of James River and between Firesteel Creek and the southern limit of this area. These exposures are usually near the water level of the larger streams, but in some cases they rise 20 or 25 feet higher.

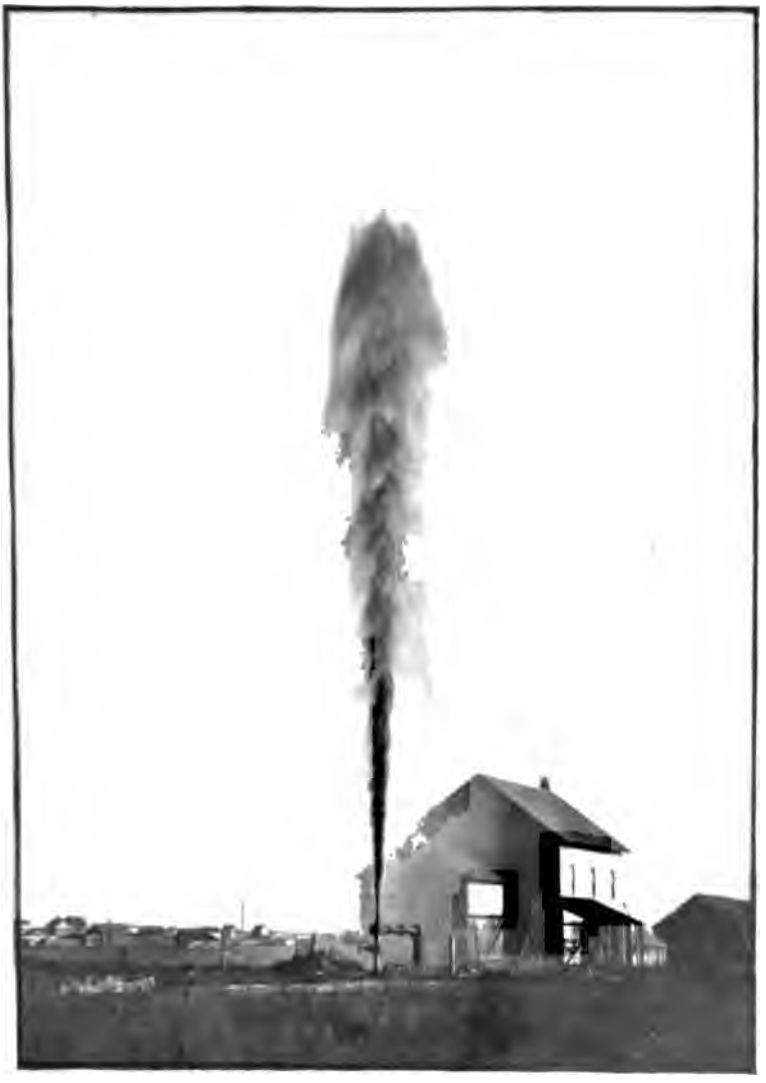
DAKOTA FORMATION.

This is the principal formation which supplies water to the important artesian wells of North and South Dakota. It nowhere comes nearer the surface than about 200 feet. Judging from well records, it consists of alternate beds of sand and sandstone, 50 to 100 feet in thickness, with masses of clay or shale interstratified between. It possibly includes in this part of the State, at least in its thickest developments, the formations which in the Black Hills were originally included in it, but which have more recently been separated under the names Fuson and Lakota, and are believed to represent the lower Cretaceous.

The Dakota formation, as exhibited in the rim of the Black Hills, is usually a brown sandstone, hard and massive below but thinner bedded above. There it is rarely over 100 feet in thickness, according to Darton. It is of medium grain, though finer and coarser grained layers are found. From material thrown out of many wells in the eastern part of the State it may usually be described as a fine-grained gray sandstone.

It extends partly over the lower portion of the ridge of red quartzite above described to an old shore line which has been uplifted to about 1,100 feet above the sea a few miles southwest of Mitchell, but owing to flexure declines toward the east and north several hundred feet before it passes beyond the eastern margin of the area under consideration. The top of the Dakota sandstone drops in all directions from this old shore line with a slope which is steep near the quartzite ridge and gradually diminishes until the formation is nearly horizontal. North of the quartzite ridge it reaches an altitude of about 675 feet at Woonsocket and less than 600 at Huron. West of Huron it seems to rise again toward the northwest in the vicinity of Wolsey. Southward from the ridge the decline is more gentle and the Dakota probably does not fall below about 800 feet above the sea, even in the remote southwest corner of the region.

The shales of the Dakota resemble those of the formations higher up, and, like them, occasionally contain calcareous concretions which may be mistaken in drill holes for limestone strata. Sometimes, also, there are concretions of pyrites large enough to cause considerable



ARTESIAN WELL AT WOONSOCKET, S. DAK.

hindrance in drilling. The different layers of sandstone are often harder near the top, and this has given rise to the expression "cap rock." Frequently the drill has to penetrate several feet of hard rock before the water-bearing strata are reached.

The various sections exhibit the character and thickness of the Dakota sandstone and associated formations. In the discussion of the sources of artesian water further light will be given on the number, thickness, and subdivisions of the sand strata in these formations. The information is mainly derived from the reports of well drillers, who unfortunately do not all agree in their interpretation of facts. Most drillers use jet drills, which mingle the drillings in such a way that it is often difficult to determine the character of the formation worked upon, and frequently also the driller is not disposed to examine with much care the character of the deposits or to measure carefully the exact position or thickness of any of the strata, points which would be of special interest to geologists. The driller is interested chiefly in the water-bearing strata, and of these only in such as produce a flow sufficient for his purpose. When asked for the history of a particular well, he is apt to remember only the depths of the flows struck and of the levels where he found the most serious obstacles. Hence it may reasonably be supposed that the deeper sandstone strata are often thicker than is reported in the logs and that the upper water-bearing sands especially are probably more numerous than usually reported.

Thickness.—Before the thickness of the Dakota can be determined it must be decided what shall be assumed as the division between this formation and the overlying Colorado. If the second important sandstone below the chalk is taken as the top of the Dakota, the formation has been penetrated to a depth of 300 or more feet in the deepest borings. It is possible that 50 or even 100 feet may be added to this before the crystalline rock is reached in the region between Huron and Woonsocket, where the formation seems to reach its greatest depth. North of Mitchell a thin and comparatively unimportant sandstone bed intervenes between the upper sandstone stratum of the Benton and the top of the Dakota.

Two wells at Huron, Beadle County, have been bored through the Dakota sandstone into the underlying quartzite and granite, and the following is a combined record:

Section of Dakota formation in city wells Nos. 3 and 4, Huron, S. Dak.

	Feet.
Sandstone, strong flow (at depth of 740 feet)	5
Sandy shale	50
Hard sandstone	12
Sand rock (much water)	65
Gray limestone ? Shale Sandstone, thin Shale	58

Section of Dakota formation in city wells Nos. 3 and 4, Huron, S. Dak.—Cont'd.

	Feet.
Sandstone	3
Black shale	125
Pebbly sand and water	9
Coarse sandstone on quartz rock or granite	30

At De Smet, on the highlands 30 miles east of Huron, a deep boring passed through a thick series of sandstone with shale intercalations, probably comprising the greater part of the Dakota formation and possibly including in its upper beds the lower portion of the Benton formation.

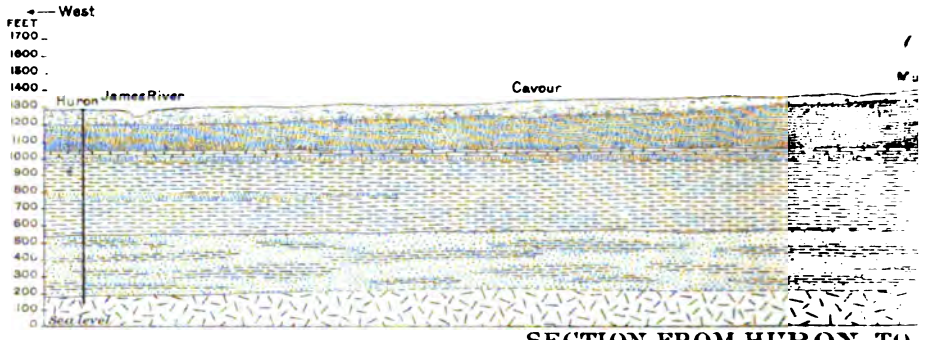
Section in lower part of boring at De Smet, S. Dak.

	Feet.
Benton?:	
Hard sandstone (at depth of 840 feet)	25
Sandstone	120
Shale	200
Dakota:	
Sandstone	271
Hard sandstone	14
Sandstone	140

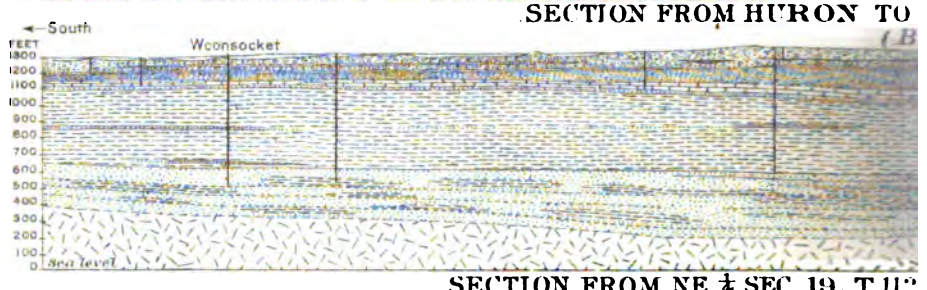
In a well 2 miles southwest of Letcher it is reported that Dakota beds have been penetrated from 500 to 683 feet without reaching their base. The material is mainly clay, including several sandstone layers with flowing water.

In the Ashmore well, southwest of Artesian, the Dakota sandstone appears to have been entered at a depth of 626 feet, and its upper members include 6 feet of sandstone and 67 feet of shale, lying on sandstone which was penetrated 2 feet.

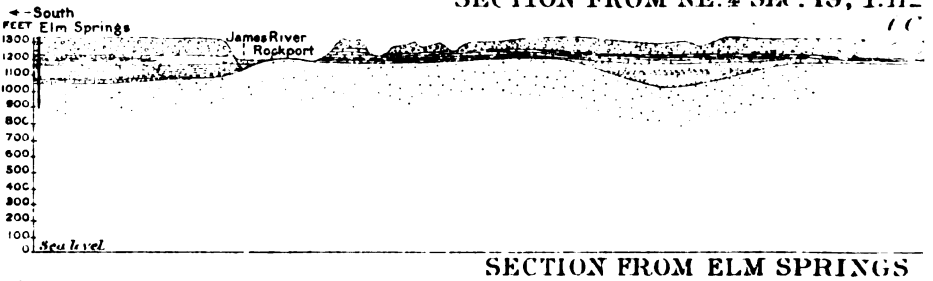
In Sanborn County many wells have reached the Dakota sandstone, and the well at the mill at Woonsocket penetrated a solid mass of it from 697 to 775 feet, but had no need to go to the base. In Davison County the thickness is variable, and the formation grows thinner and disappears entirely in an irregular area on the higher slopes of the underground ridge of quartzite south and southwest of Mitchell. At Mitchell it appears to extend from 440 feet to the quartzite at 540 feet, comprising 39 feet of sandstone above and 11 feet of sandstone at base, separated by 50 feet of shale. In the Smith & Davison well, 4 miles southwest of Mitchell, in a valley in the buried quartzite ridge, the formation is represented by 40 feet of sandstone and 10 feet of shale, lying on quartzite of which the top is at a depth of 475 feet. At Ethan there are only 8 feet of sandstone on the quartzite, and this may belong to the Benton. In the Lowrie well, northwest of Ethan, the Dakota appears to be 92 feet thick. It lies on quartzite at a depth of 477 feet, and consists of a top member of sandstone about 30 feet thick and a lower series of shales. The formation thickens rapidly to the west and south, and in the J. K. Johnson well, 3 miles due north of Mount Vernon, where the underlying bed rock was not reached, the section is as follows:



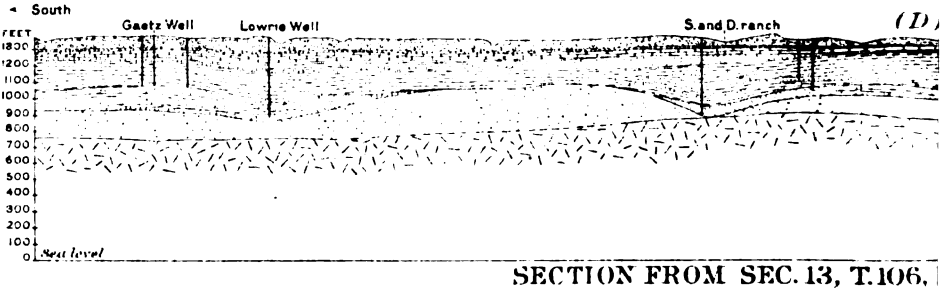
SECTION FROM HURON TO



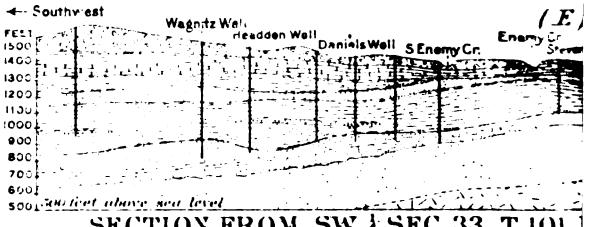
SECTION FROM NE. ¼ SEC. 19, T. 112



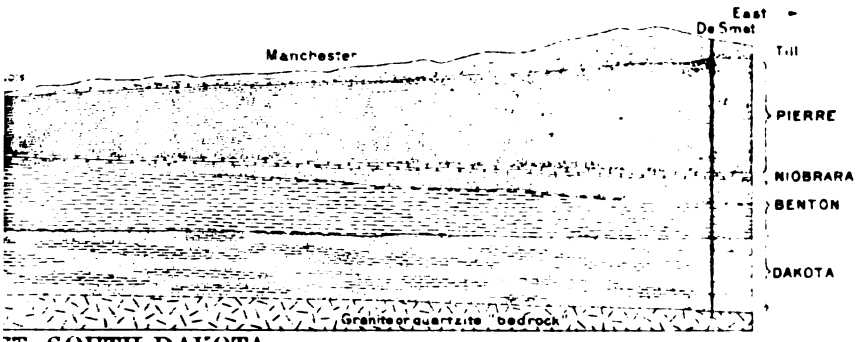
SECTION FROM ELM SPRINGS



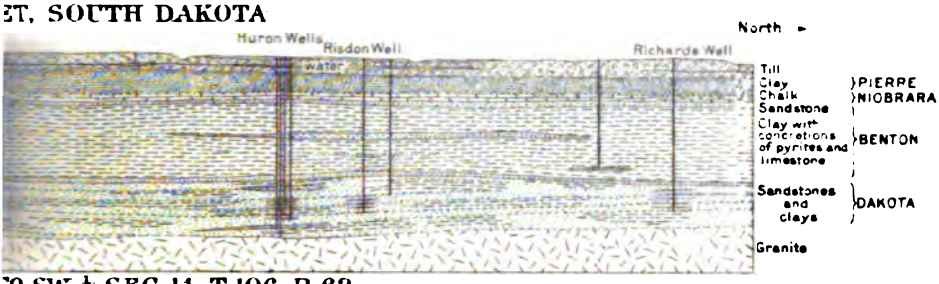
SECTION FROM SEC. 13, T. 106,



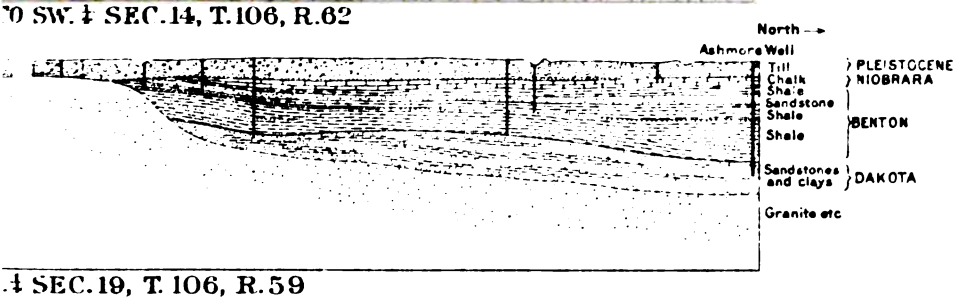
SECTION FROM SW. ¼ SEC. 33, T. 101,



ET, SOUTH DAKOTA



0 SW. ¼ SEC. 14, T. 106, R. 62



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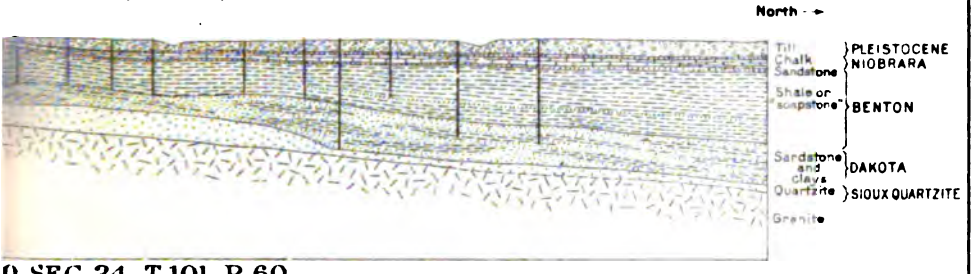

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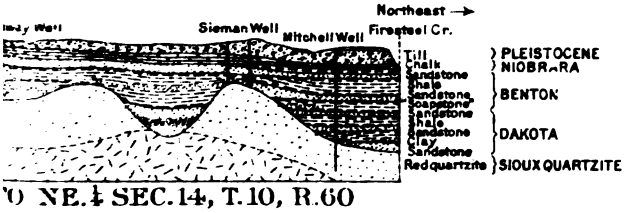


0 NE. ¼ SEC. 14, T. 10, R. 60

JULIUS BIEN & CO. LITH N Y

Section of Dakota formation north of Mount Vernon, S. Dak.

	Feet.
Sandstone (at depth of 350 feet)	70
Shale	50
Sandstone	36
Shale	90
Sandstone	44

In eastern Aurora County several wells penetrate the formation a hundred feet or more and find alternate beds of sandstone and shale. The well at Plankinton is reported to have reached granite at 756 feet, and the overlying Dakota appears to comprise 218 feet of beds (from a depth of 538 feet), reported to consist largely of shale, with several thin sandstone bodies.

From the Storla well, in the southwest corner of the county, are reported beds, apparently Dakota, from a depth of 457 to 760 feet, as follows:

Partial section of Dakota formation in Storla well, Aurora County, S. Dak.

	Feet.
Sandstone	13
Shale	60
Sandstone	5
Shale	30
Sandstone	10
Shale	45
Sandstone	10
Shale	110
Sandstone	20

In the Dougan well, 4 miles northeast of the Storla well, there were 23 feet of sandstone on 60 feet of shale lying on sandstone.

In the Bartow well, 3 miles northeast of Plankinton, the following beds are reported:

Partial section of Dakota formation in Bartow well, near Plankinton, S. Dak.

	Feet.
Sandstone (at depth of 455–516 feet)	61
Shale	104
Sandstone	Few
Shale, etc	133
Sandstone	2

In the Raesley well, 4 miles southeast of Plankinton, the section is as follows:

Partial section of Dakota formation in Raesley well, near Plankinton, S. Dak.

	Feet.
Sandstone (at depth of 477 feet)	12
Shale	116
Sandstone	5
(?)	24
Shale	45
Sandstone	39

The formation is absent in the central and southern portion of Hanson County, but in its northern and northeastern part it has been penetrated by several deep wells and found to consist of 20 to 53 feet of sandstone lying on granite or quartzite. Apparently it thickens rapidly northward under the western part of Miner County and in the southeastern part of Sanborn County.

Fossils.—The Dakota formation is considered to be a fresh-water deposit from the fact that fossils occur rarely and that a few distinctly fresh-water species have been found in it.

A species of *Goniobasis* was obtained in quantity from wells around Esmond at a depth of 785 feet, or about 30 feet above the main flow. Mr. T. W. Stanton, to whom the specimens were submitted, declares them identical with a species found in the Dakota of Jefferson County, Nebr.[a]

BENTON FORMATION.

The Benton formation consists mainly of dark shales, with thin beds of sandstone and limestone. In an area of considerable extent in Sanborn and adjoining counties the Niobrara chalkstone lies on a bed of sandstone 10 to 85 feet thick, sometimes with a thin intervening mass of shales. This was formerly supposed to be a portion of the Dakota sandstone, for it often contains considerable water, sometimes under pressure, but it has been found to be underlain by a thick mass of typical Benton clay containing marine fossils. This upper sandstone outcrops at several localities along Firesteel, Enemy, and Twelvemile creeks, and along James River at intervals from north of Mitchell to beyond Elmspring. The most northern point where it appears on the surface is in the bottom of the valley of James River, in sec. 22, T. 104 N., R. 60 W. A mile or two farther south it is exposed at several points along Firesteel Creek, particularly near the crossing of the railroad north of Mitchell, and at points as far west as sec. 36, T. 104 N., R. 62 W. The outcrops near the railroad rise about 20 feet above the stream, or to an altitude of 1,280 feet above sea level. It is exposed also along Enemy Creek south of Mitchell, and between that point and James River, and forms low cliffs along the river below Rockport, rising 30 to 50 feet above the stream. It is a rusty-brown sandstone, usually hard and of dark color on the surface, but softer below, varying much in character, sometimes being coarse and containing small pebbles and at other times extremely fine grained. It frequently shows oblique lamination in strata 3 or 4 feet in thickness.

No fossils have been found in these outcrops except sharks' teeth, which in some places are very numerous. Traces of wood and leaves have also been found at a few points. Its upper surface is uneven, but this is not merely due to erosion. It seems rather to have been

[a] Proc. U. S. Nat. Mus., vol. 17, 1894, p. 135, pl. viii, fig. 10.

RISDON ARTESIAN WELL, NEAR HURON, S. DAK., THROWING A 10-INCH STREAM TO A HEIGHT OF 12 FEET.

caused by unequal deposition, which may correspond to sand reefs, or possibly dune action, although there are no distinct traces of the latter.

The shale which overlies the upper sandstone of the Benton in part of the area attains a thickness of 50 feet in its greatest development. It appears not only in wells, but in a few exposures in secs. 25 and 26, T. 103 N., R. 60 W., near the railroad, on opposite sides of the ravine leading into James River.

Fossils characteristic of the Benton have been reported in wells on the east side of James River, near the south line of Hanson County, more than 100 feet below the surface, and in several wells in the northeastern part of Hanson County. In sec. 36, T. 104 N., R. 62 W., there are several feet of dark clay intervening between the chalkstone and the sandstone in an exposure on the south side of Firesteel Creek. Wells in that vicinity, also those near and south of Plankinton, disclose a dark clay next below the chalk, which is probably the same member.

Below the sandstone lie about 200 feet of gray and black shales indistinguishable from similar deposits in other formations of the Cretaceous. There occurs in this shale, in the vicinity of the quartzite ridge near Mitchell, another stratum of sandstone which seems to be thicker, more porous, and therefore more water bearing. North and west of Letcher it apparently becomes very thin and is often overlooked by drillers. Nevertheless, when carefully looked for it may usually be found, and will furnish a meager supply of water.

Thickness.—The upper surface of the Benton, as has already been stated, rises nearly 1,300 feet above sea level in the vicinity of Mitchell, and north and south of Plankinton it rises higher than 1,300 feet, judging from the reports of wells. It rarely falls below 1,000 feet in the region around Huron. Its thickness, therefore, may be estimated to be 350 to 400 feet, or nearly double the estimate given by Doctor Hayden from his observations along Missouri River.

Fossils.—Besides the sharks' teeth and traces of vegetation already mentioned, special note should be made of a stratum of fossiliferous limestone discovered in several wells in the vicinity of Woonsocket. Though reported from several wells, our most definite knowledge comes from a well 2 miles north of Woonsocket. From a break in the pipe, which was afterwards proved to be 580 feet below the surface, fragments of a fossiliferous limestone were frequently thrown out. These were submitted to Mr. T. W. Stanton, of the Survey, and he reports that at least three different kinds of fossils were represented, of which one was a small *Nucula* with striated surface that may be the young of *N. cancellata*, M. and H. Another is possibly a young *Mactra*, and the third, the most common form, was probably a *Lucina*. The specimens were too imperfect to permit more definite determination. They were found 250 feet below the chalkstone and about 100

feet above the main flow. These are distinctly marine in character, and hence form a strong reason for considering this stratum as part of the Benton. In the well at Farwell sharks' teeth were found in large number in the shales 400 feet below the surface. In the Ashmore well, southwest of Artesian, fossils were obtained a short distance above the top of the water-bearing Dakota sandstone, comprising a *Mactra* and a *Fasciolaria*.

From the black clay above the sandstone north of Mount Vernon a saurian vertebra about 4 inches in length was obtained.

NIOBRARA FORMATION.

The most characteristic feature of this formation is the chalkstone, which merges into calcareous clay and shale. It is of drab color, but on weathering becomes white, cream-colored, or often a light-straw color. As the formations both below and above are clay, it follows that the limitations of this formation can not be very sharply defined where the more impure deposits are dealt with. The recognition of this formation is especially difficult in wells, for where the chalk has not been exposed to atmospheric action it presents a leaden color, closely resembling the gray clays of the Pierre and Benton. Many well drillers do not recognize it north of the vicinity of Woonsocket.

The most definite distinction between the chalkstone and the clay is that when pulverized the chalkstone does not become plastic and sticky like the clay, but behaves more like a fine sandstone, from which, however, it is usually readily distinguished by its softness and lack of grit. From the evidence obtained it remains doubtful whether real chalkstone extends into the northern part of the area around Huron or into the western part about Plankinton, farther south. Perhaps the strongest evidence of its presence there is the fact that valuable pump wells are obtained at points where we should expect to find it. These wells are commonly said to derive their water from "soapstone," but it is difficult to see how that can be, for clay shales are commonly very impervious. It is well known that similar waters are obtained from chalkstone in the shallower soft-water wells west and southwest of Mitchell. Moreover, the wells said to be from the soapstone in the Huron quadrangle are soft-water wells like those from the chalk farther south. In the southern half of the area the chalk is often conspicuously developed and outcrops in many exposures. They appear at intervals of a mile or so along Firesteel Creek from its mouth to a point north of Mount Vernon. In some cases the chalkstone and sandstone of the uppermost Benton appear in the same vertical section. For 5 or 6 miles west of the railroad bridge over Firesteel Creek, sections of chalkstone and sandstone alternate at about the same level, indicating unconformity. Whether this is due to erosion or flexing or to some irregularity in deposition has not been surely determined, but it is probably the latter. Exposures of

A. UPPER BENTON SANDSTONE ON FIRESTEEL CREEK, IN T. 104 N., R. 61 W.

B. ARTESIAN SPRING IN T. 104 N., R. 60 W.

chalkstone appear along a channel of Enemy and Twelvemile Creek in T. 102 N., R. 61 W.; also on the lower portion of Twelvemile Creek east of Ethan. It is exposed also along James River near Rockport, at the mouth of Redstone Creek, and above the mouth of Firesteel Creek. In several places it lies only a few feet above the Algonkian. Many feet of it are found in wells 3 or 4 miles north of Alexandria, but, in general, the summit of the quartzite ridge from Alexandria eastward seems not to be overlain by it.

Thickness.—Over most of the area under consideration the Niobrara has been much thinned by erosion. In a strip running through the Alexandria quadrangle from Farmer northward wells fail to show the presence of chalkstone. Instead, there is a deposit of sand, presumably of Pleistocene age. The outlines of this have not been very clearly determined, but it seems to be several miles in width. East of this strip it would appear from the reports of some well drillers that the chalkstone comes in as if in the form of a buried escarpment. It seems not unlikely that the pre-Glacial course of James River may have been in that direction.

The thickest deposits of chalkstone yet reported are in the southwestern part of the Mitchell quadrangle, where a thickness of more than 200 feet is given. This may include some shale below. In some of the sections in the northern part of the Huron quadrangle what has been reported as gray shale has been indicated in the figures as chalkstone. It is not unlikely that, if carefully examined, the chalkstone of a bluish gray color might be found as thick in that portion as anywhere. At any rate, the beds including clay, which belong properly to the Niobrara, may be considered to be at least 200 feet in thickness.

Fossils.—The chalkstone frequently contains teeth and scales, mostly of bony fishes, although sharks' teeth are also found. Occasionally quite perfect specimens of bony fishes have been found.

The most common fossil is a small oyster about an inch in length, called *Ostrea congesta*, which is especially characteristic of the Niobrara formation. These fossils occur in thin layers of limestone and are frequently clustered on fragments of large shells, either *Pinna* or *Inoceramus*.

PIERRE SHALE.

This formation consists almost entirely of dark, plastic clay or soft shale, with occasional calcareous concretions. It is very uniform in character, but varies slightly in color. It underlies the drift in the northern half of the area to which this report relates, but has been eroded from the uplifted area about Mitchell. It dips northwestward and increases in thickness in that direction and in the northern portion of the area, especially in the highlands between Huron and De Smet, it is several hundred feet thick.

TERTIARY GAP.

No traces of the latest Cretaceous, or Laramie, nor of the Tertiary beds found farther west have been recognized in this area. It is not improbable that the thin edge of the Miocene may have extended over this region, but the Pliocene and early Pleistocene erosion which excavated the James River Valley before the advent of the ice must have removed them.

QUATERNARY.

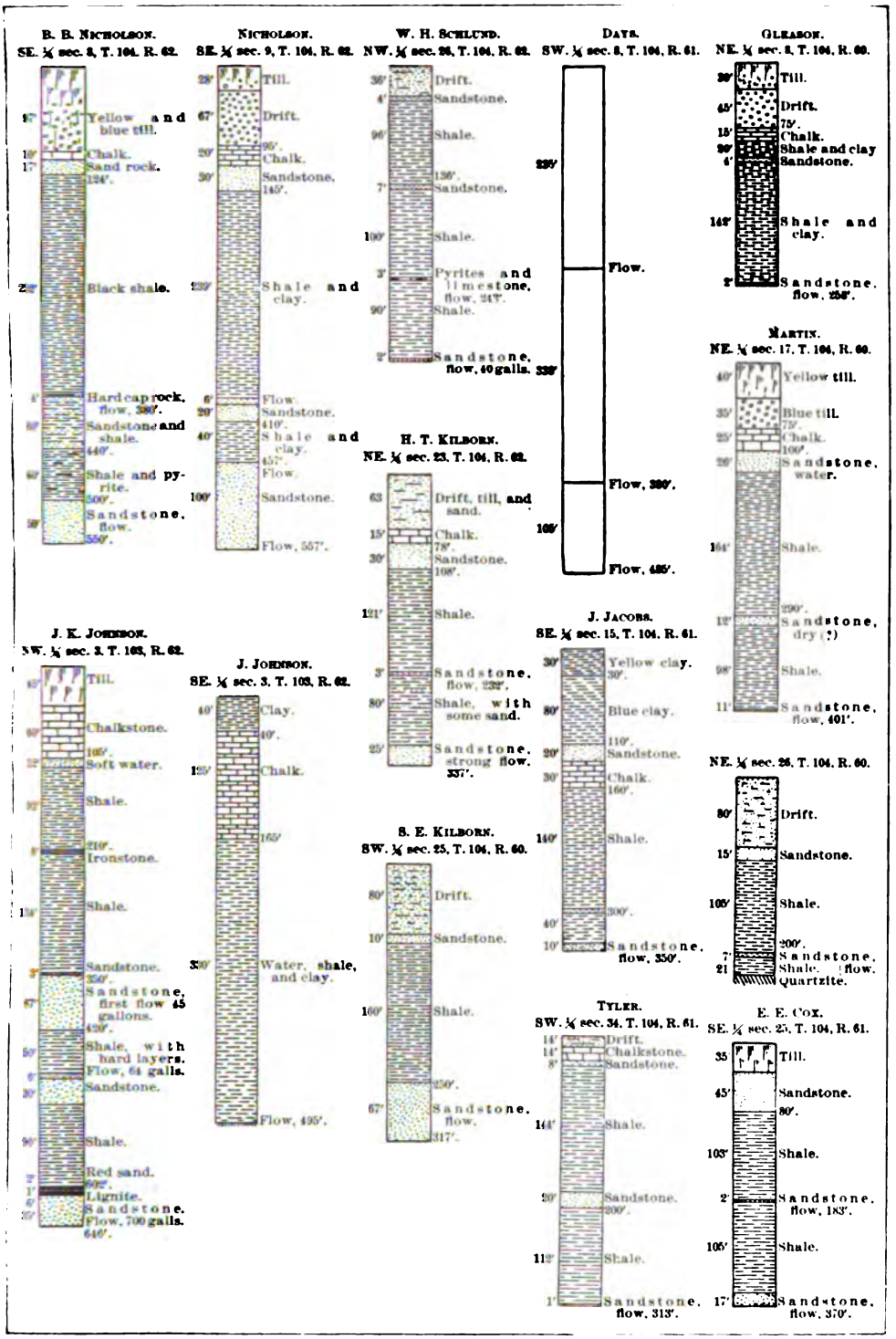
As already stated, the earlier Quaternary was an epoch of erosion. James River Valley at that time had attained approximately its present form and relative altitude, but the peculiar distribution of certain water-bearing strata below the till north of the quartzite ridge may have taken place in that period. The fact that from some wells in that region pieces of peat and numerous fresh-water shells have been thrown up strongly suggests that they came from a pre-Glacial marsh deposit which may have been connected with the flood plain of James River of that time.

With the exception of these beds most of the formations we have considered thus far have been of marine origin. In the remainder of the Quaternary deposits there is a marked contrast, not only in the cause and method of deposition, but also in the way in which they overlie all earlier formations without respect to altitude.

GENERAL DESCRIPTION OF GLACIAL ACTION.

To understand the conditions under which the Pleistocene deposits were laid down, it is necessary to know something about ice sheets, such as are now found in Greenland or Alaska. The beginning of a glacier is due to the accumulation of snow. As it accumulates from year to year, partial melting and the pressure of superincumbent material compresses it into ice and it commences a slow, flowing movement down a valley. The ice moves like the river, except that the motion is sometimes independent of local valleys or channels, and is very much slower. Its form is more lobate, with the breadth often nearly as great as the length, and the movement is everywhere toward the edge.

The limit of the ice is determined by the relation of the melting to the velocity. If these influences are just balanced—in other words, if the ice at any point melts as fast as it moves forward—the edge will remain constant, and as the ice transports much gravel and clay, a ridge or moraine will be formed at the edge. If the ice moves more rapidly because of rapid accumulation of snow at the fountain head, or if the melting diminishes, as in winter or in a succession of cool and cloudy seasons, the ice will advance, overriding the moraine, if such has been formed. If the converse in either of these cases is true, the edge of the ice will recede, and if continued sufficiently the



LOGS OF WELLS IN DAVISON COUNTY, S. DAK.

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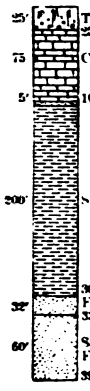
C. ARLAND.
SE. ¼ sec. 15, T. 103, R. 62.



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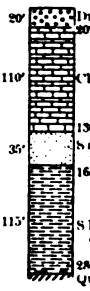

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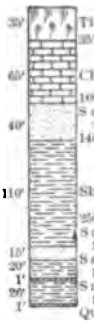

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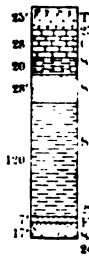

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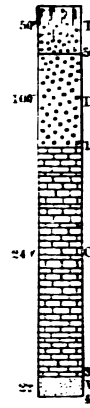

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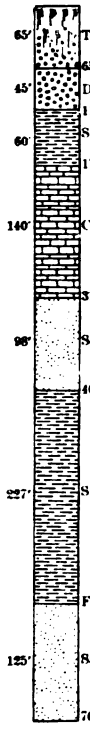

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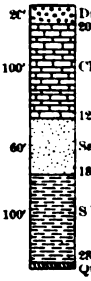

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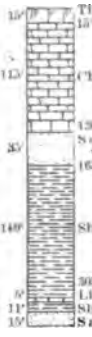

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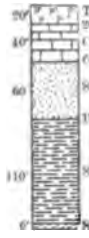

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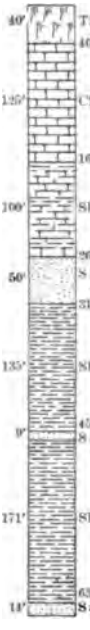

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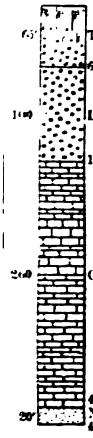

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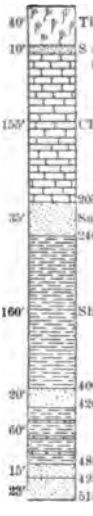
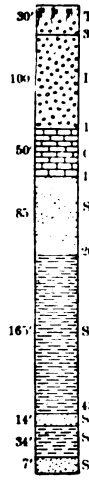


SE. ¼ sec. 15, T. 101, R. 61.



LOGS OF WELLS IN DAVISON COUNTY, S. DAK.

ice will gradually withdraw until it entirely disappears. Such has been the case with the vast ice sheets which comparatively recently covered this region.

As the ice proceeds the débris under the thicker central portions will be pushed along, producing scratches and grooves upon the underlying rocks, but toward the edge, where the ice sheet is thinner because of rapid melting above and below, the débris below, consisting of bowlders and clay, will cease to be moved, and, like the sediment of a river when the current slackens, will build up a bar. Moreover, the material carried in the lower part of the ice sheet will gradually be dropped and become more or less assorted by the waters which result from the melting of the ice. The resulting deposits will be of fluvial character where drainage is free, and lacustrine where escape is hindered. In the latter case especially portions of the ice mass will become detached and buried in the débris and their melting will be delayed until after the surrounding material has become permanently settled, so that eventually pond holes, and even lake beds of considerable size, will result.

From this sketch of the glacial action one can readily understand how the first advance of the ice is preceded by a flooded condition of all streams. In case they flow from the ice, their waters will be swollen by the melting ice and their sediment increased by the mud and sand contributed by the glacier. In case their course is toward the ice sheet or parallel with the edge, they will become dammed and changed temporarily into lakes in which the material from the ice will be deposited.

From this explanation also it may be seen that the pre-Glacial surface will be more or less covered with a layer of sand or gravel before the bowlder clay or regular glacial till is deposited upon it.

It is also seen how the pre-Glacial surface divides the course of the ice sheet and decides the character, in a measure, of the deposits laid down by it. The axis of the ice flow will coincide with the central portion of the pre-Glacial valley, and over it the ice will be thickest and its surface most elevated. The divides between the valleys and other elevated points will be the last to be covered by the ice and the first to be uncovered. From this fact it might be expected that the glacial deposits are thinner there, but such is not often the case, because of the tendency for the deposits to accumulate more rapidly under the thinner portions of the ice, as before explained. These facts are well illustrated in this area.

TILL OR BOWLDER CLAY.

This deposit is the accumulation formed underneath the ice, as has already been explained. It is generally of unstratified character, being a heterogeneous mixture of sands, clay, gravel, and occasional bowlders. It is usually distinguished from ordinary stream deposits

by its unassorted character and the absence of stratification. It frequently contains pockets or lenses of sand of considerable extent, which is often stratified. In some cases these are portions of the accumulations of subglacial streams, which have become broken up and buried in the till. They sometimes contain abundant water, but are often dry and filled with air, which is sometimes under pressure. The ingredients of the till vary in different localities according to the rocks over which the glacier has passed. In this area the clay comprises perhaps 90 per cent of the whole.

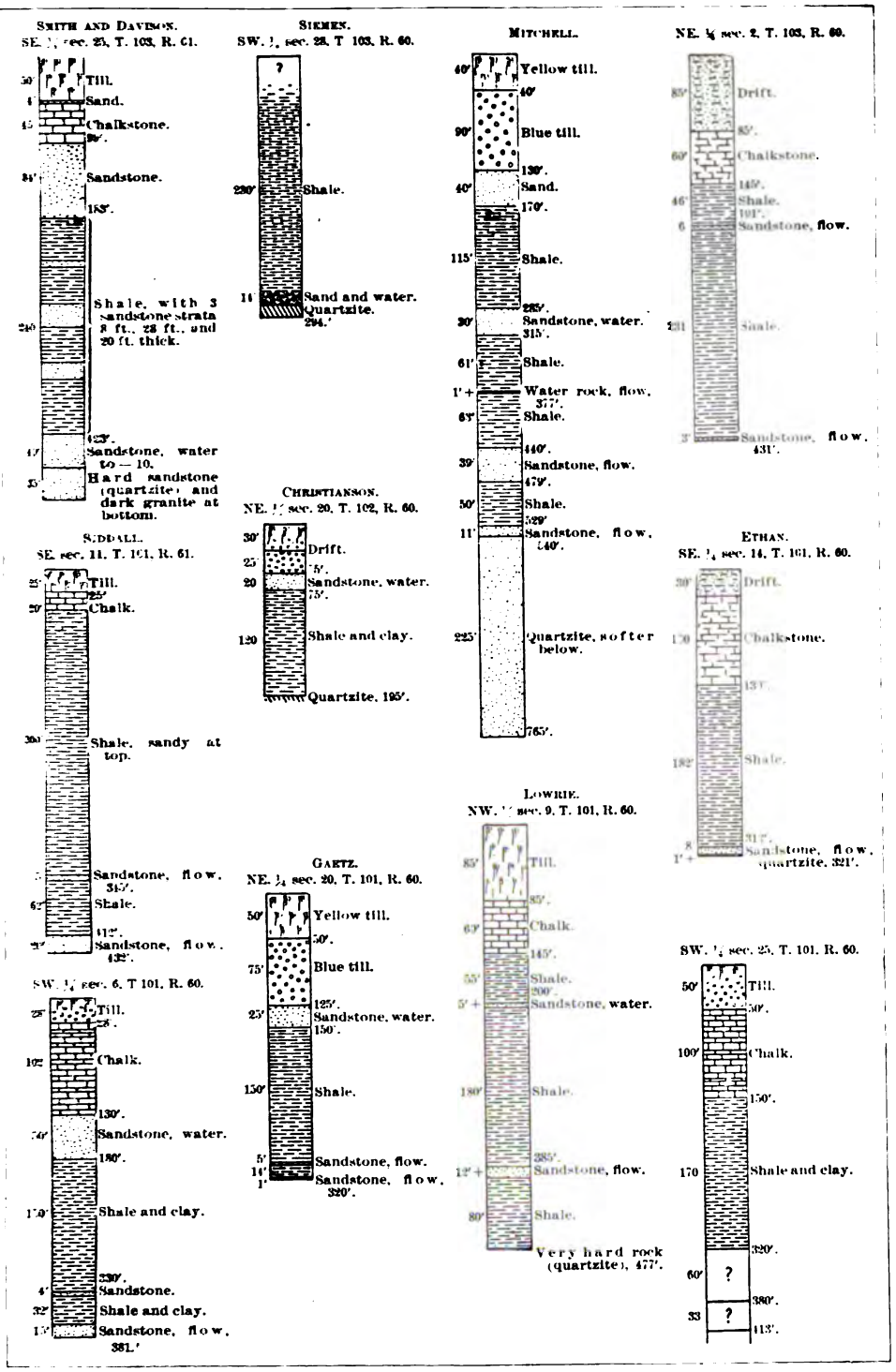
Surface of the till.—The surface of the till in lower places is more or less covered with silt, laid down by the waters escaping from the ice soon after its deposition, and also by the wash in more recent times, especially upon hillsides. In some localities, also, considerable silt has been deposited from the atmosphere; but both of these influences have not modified the till of this region to any great extent, except in the lower portions bordering James River, of which mention will be made in a later section. The surface of the till in general may be said to be much more even in this area than is frequently the case in other regions. This is mainly the result of its having been deposited far within the principal moraines upon a surface already smoothed by former and more vigorous ice sheets.

The till of this region exhibits the usual basin and swell surface, but in a very mild degree. Few of the basins are of sufficient extent to hold water through the summer time. As a whole, however, they have a most important influence in retaining the surface water. When snow melts, or after rains, they hold the water so that over much of the surface very little of the rainfall escapes into the streams.

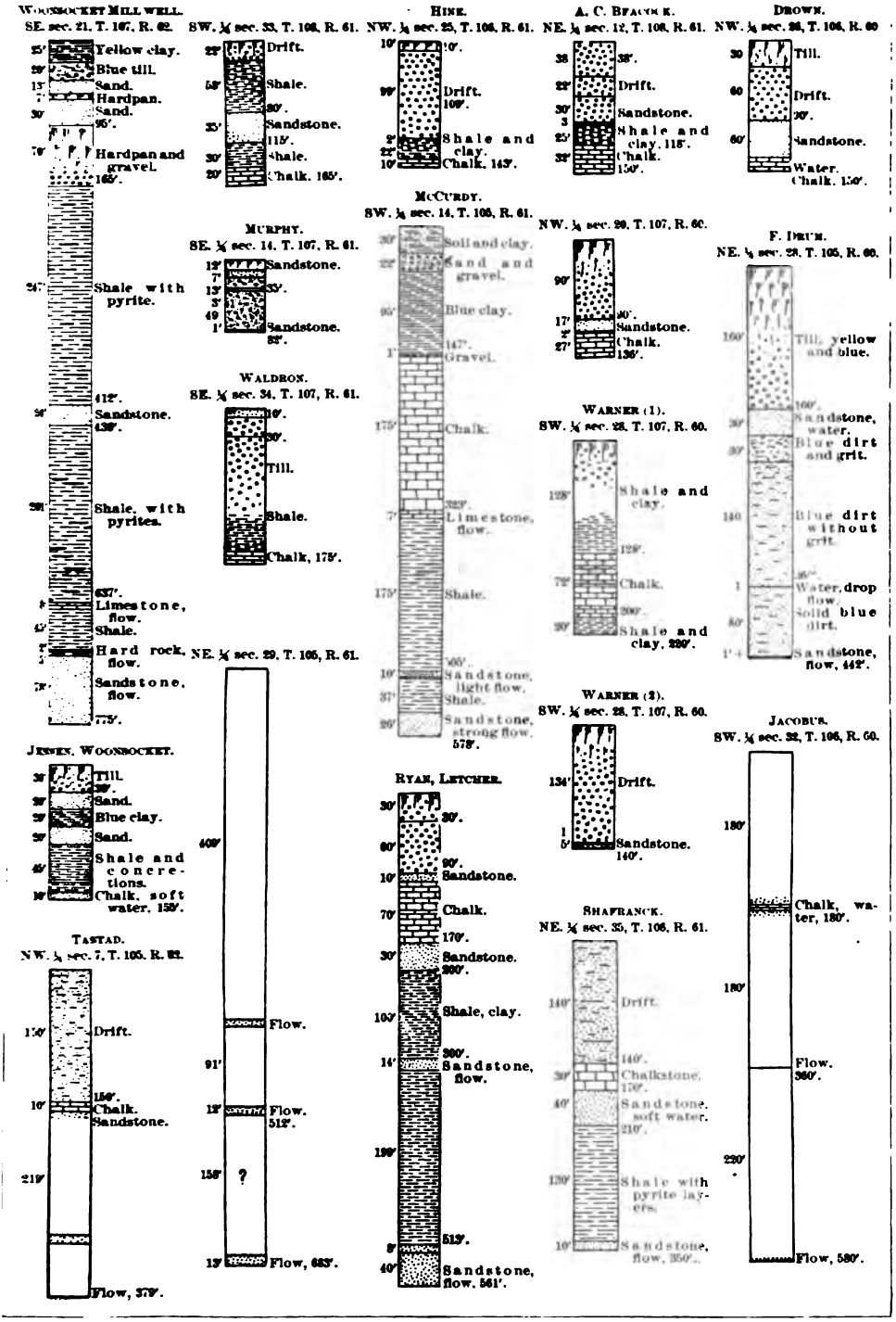
An area around Huron, covering perhaps 100 square miles or more, seems to have been, during the recession of the ice, a shallow lake. So, also, at an earlier stage there was an area of similar character and equal extent east of Woonsocket, and also another northeast of Mitchell. All of these will be discussed more in detail under the heading "Ancient channels."

Thickness of the till.—The average thickness of the till can be roughly estimated to be about 75 feet over the whole area. Southwest, west, and south of Mitchell, within a radius of 10 or 12 miles, the thickness is often less than 50 feet. This includes the region in which the Cretaceous strata outcrop. So, also, in the valleys of all the principal streams the thickness of the till is often less than 20 feet. On the contrary, in the elevated region about De Smet depths of 200 or 250 feet may occur, although but few drillings have been made to show this fact. Southward from that region the till gradually diminishes in thickness, until in the vicinity of Spencer it is generally less than 50 feet. In the region south of Plankinton, also, wells show a frequent thickness of 150 feet, while some attain over 200 feet.

In the flat lands east of Woonsocket the depth of the till is nearly



LOGS OF WELLS IN DAVISON COUNTY, S. DAK.



LOGS OF WELLS IN SANBORN COUNTY, S. DAK.

100 feet, and this thickness prevails generally under the morainic belt north of Sand Creek and Redstone Creek. Southeast of Wolsey, in the flat country, the till usually has a thickness of 60 or 70 feet; it probably has about the same thickness in the flat lands around and east of Huron.

In considering these statements it should be remembered that the separation of the till from the underlying clay of the Cretaceous is often very indistinct. Well drillers very frequently do not distinguish the two formations; hence the reports upon which the preceding statements have been made are few and far between. In the region near the Cretaceous outcrops no water is found between the till and the underlying chalkstone or sandstone. Hence in drilling wells the till is not looked for in such portions of the region. Elsewhere, also, in this area, more frequently than farther south, there seems to be an absence of the sand stratum which separates the till from the underlying clays, and even in some cases where sand is found it is so fine that it can not be utilized for water supply. It is nearly impossible to separate the fine sand from the water, even when the casing is covered with very fine-meshed screen.

Another consideration also tends to produce confusion, especially in the northern part of the area. There exist in the upper part of the Cretaceous one or two water strata, which join the bottom of the till at a very small angle. Hence, where the thickness of the till is estimated from the depth of wells, by an oversight the measurement may extend below the till to the Cretaceous clays. This danger is especially strong in the northern half of the area.

Another source of error, against which it is impossible to guard, is the irregular thickness of the till in some localities. Two wells sunk only a few rods apart are frequently reported to find the bottom of the till at very different depths. Sometimes such reports may arise from the sand pockets which we have already referred to as occurring in the till, but in some cases the statements refer to the striking of soapstone, or Cretaceous clay. In this case there could be little doubt that the bottom of the till had been reached. On the whole, therefore, the statements concerning the thickness of the till must be taken as only very general estimates.

MORAINES.

The till sheet already described is sometimes called the ground moraine, but the moraine spoken of here is always the system of hills believed to have been heaped up around the edge of the ice sheet. It is sometimes more definitely called a terminal moraine. Such hills are recognized more by their relative position than by any one characteristic which they bear individually. Each one is usually lengthened parallel with the course of the system, abounds in bowlders, and has somewhat abrupt sides, but all of these characteristics are not always found together.

As a system, the moraine usually presents these stony, knobby hills mingled confusedly with circular and winding basins which often contain water, but sometimes both basins and hills are very faintly developed, so that the whole constitutes a broad swell. The moraine is traversed here and there by valleys through which water escaped from the ice sheet. These may be of trifling size or many rods in width, and cut down through the whole height of the moraine.

The height of the moraine at any one point is due to several conditions:

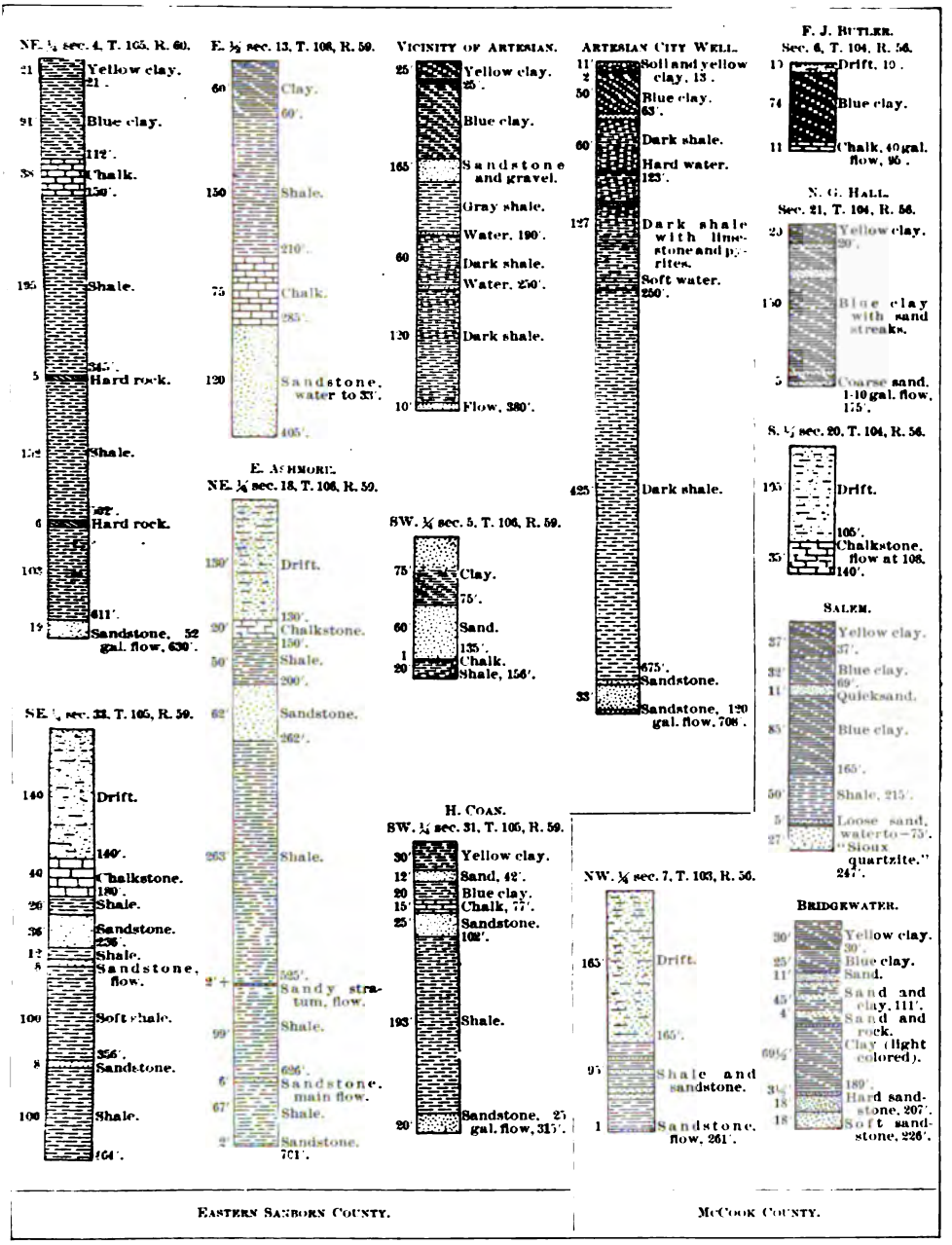
1. The relative height of the pre-Glacial surface.
2. The amount of material pushed forward by the ice at that point, which depends again on the transporting power of the glacier and the time its edge remained stationary.
3. The amount of material washed away.
4. The consistency of the material—an excess of clay or sand tending to produce a low and even moraine, while excess of stony material is favorable to the formation of a high and rugged moraine.

The moraines of this area are mapped on Pl. III; consequently it will be unnecessary to speak definitely of their location. With few exceptions they are not conspicuous features. Generally they consist of low, broad swells, showing the usual surface of the till, except that occasional peaks which rise more or less abruptly 15 or 25 feet above the surrounding surface are scattered over the general level. The swell may have a height of 20 or 30 feet above the till, into which it insensibly merges. The intervening depressions and basins are also more numerous and larger than elsewhere. Moreover, the moraines usually present a large number of bowlders, beds of gravel, and other marks of abundant and free-flowing waters.

In the area under consideration are portions of two principal moraines, both of which present three or four minor subdivisions. The moraines belong to the Wisconsin stage of the Glacial epoch and are commonly known as the Second, or Gary moraine, and the Third, or Antelope. The former name is derived from the town of Gary, S. Dak., where it was first studied, and the second from a ridge in Minnesota Valley. In general it may be stated that these moraines were formed as portions of loops extending southward across the axis of James River Valley. Previous to their formation the ice had covered the whole area to a depth of several hundred feet, and pushed south and west to the vicinity of Missouri River, where it rested for a time and formed the First, or Altamont moraine. Hence the moraines in this area were formed on a fairly smooth and level surface. For the same reason, also, they are less irregular and prominent.

These loops show higher hills upon the southwest side, while upon the southeast and east they are sometimes not easily traced.

The Gary moraine enters the west boundary of the area north of Firesteel Creek, near the north boundary of Jerauld County. It is


NE. ¼ sec. 4, T. 105, R. 60.
21 Yellow clay. 21'.
91 Blue clay. 112'.
38 Chalk. 150'.
195 Shale. 345'.
5 Hard rock. 350'.
152 Shale. 502'.
6 Hard rock. 508'.
103 611'.
19 Sandstone. 52 gal. flow. 630'.
SE. ¼ sec. 33, T. 105, R. 59.
140 Drift. 140'.
40 Chalkstone. 180'.
26 Shale. 206'.
36 Sandstone. 236'.
12 Shale. 248'.
8 Sandstone, flow. 256'.
100 Soft shale. 356'.
8 Sandstone. 364'.
100 Shale. 464'.
E. ½ sec. 13, T. 108, R. 59.
60 Clay. 60'.
150 Shale. 210'.
75 Chalk. 285'.
120 Sandstone, water to 33'. 405'.
E. ASHMORE.
NE. ¼ sec. 18, T. 106, R. 59.
130 Drift. 130'.
20 Chalkstone. 150'.
50 Shale. 200'.
62 Sandstone. 262'.
263 Shale. 525'.
2+ Sandy stratum, flow.
99 Shale. 626'.
6 Sandstone, main flow.
67 Shale. 693'.
2 Sandstone. 701'.
VICINITY OF ARTESIAN.
25 Yellow clay. 25'.
165 Blue clay. Sandstone and gravel. Gray shale. 190'.
60 Water. 190'. Dark shale. 250'.
120 Water. 250'. Dark shale. 370'.
10 Flow. 380'.
SW. ¼ sec. 5, T. 106, R. 59.
75 Clay. 75'.
60 Sand. 135'.
1 Chalk. 136'.
20 Shale, 156'.
H. COAN.
SW. ¼ sec. 31, T. 105, R. 59.
30 Yellow clay. 30'.
12 Sand, 42'.
20 Blue clay. 62'.
15 Chalk, 77'.
25 Sandstone. 102'.
193 Shale. 295'.
20 Sandstone, 25 gal. flow, 315'.
ARTESIAN CITY WELL.
11 Soil and yellow clay, 13'.
2
50 Blue clay. 63'.
60 Dark shale. 123'.
Hard water. 123'.
127 Dark shale with limestone and pyrites. 250'.
Soft water. 250'.
425 Dark shale. 675'.
Sandstone.
33 Sandstone, 120 gal. flow, 708'.
NW. ¼ sec. 7, T. 103, R. 56.
165 Drift. 165'.
95 Shale and sandstone. 260'.
1 Sandstone, flow, 261'.
F. J. BUTLER.
Sec. 6, T. 104, R. 56.
17 Drift, 19'.
74 Blue clay. 93'.
11 Chalk, 40 gal. flow, 96'.
N. G. HALL.
Sec. 21, T. 104, R. 56.
23 Yellow clay. 23'.
150 Blue clay with sand streaks. 173'.
5 Coarse sand, 1-10 gal. flow, 175'.
S. ½ sec. 20, T. 104, R. 56.
105 Drift. 105'.
35 Chalkstone, flow at 108. 140'.
SALEM.
37 Yellow clay. 37'.
32 Blue clay. 69'.
11 Quicksand. 80'.
85 Blue clay. 165'.
50 Shale, 215'.
5 Loose sand, water to—75'.
27 "Sioux quartzite." 247'.
BRIDGEWATER.
30 Yellow clay. 30'.
25 Blue clay.
11 Sand. 66'.
45 Sand and clay, 111'.
4 Sand and rock. 115'.
69½ Clay (light colored). 184½'.
3½ Hard sandstone, 207'.
18 Soft sandstone, 225'.
18

EASTERN SANBORN COUNTY. McCOOK COUNTY.

LOGS OF WELLS IN JAMES RIVER VALLEY, SOUTH DAKOTA.

there usually developed in an irregular ridge, rising more than 100 feet above the valley of the stream outside, or west. It continues southward along the east side of Firesteel Creek, soon becoming a low swell, and so continues across the main channel of Firesteel Creek into Davison County. Here it follows a tributary from the south, which connects by a valley with the upper portion of Enemy Creek, and again with the upper part of Twelvemile Creek. The outer margin of this moraine still follows this channel in a southeasterly direction till it crosses the north line of Hutchinson County east of the ninety-eighth meridian. This outer member does not reappear in this area until it crosses the northeast corner in a north-northwest direction, west of De Smet. Branching from this, near the north line of Davison County, is the second member, which passes more to the east along the main channel of Firesteel Creek and James River, crossing the latter near the south boundary of the area. This does not appear again until it is found in connection with the first member near De Smet. A third member leaves the other two southwest of Woonsocket, and takes a course approximately parallel with the last, passing south and a little southeast of Mitchell, forming a short but high reentrant angle just east of James River, northwest of Bard, thence continuing along the east side of James River until it is lost in the plain east of Rockport. Its eastern limb reappears near Epiphany and continues north in a low ridge west of Wolf Creek and west of Howard, keeping the same direction till it joins the older members west of De Smet. A fourth member, rather more clearly defined in its western portion than the last, leaves the preceding member northwest of Woonsocket, and about 3 miles west of that place is developed into a high, narrow ridge, known as the "Pony Hills." It diverges still more from the former members so that it crosses the railroad in scattered knolls about 5 miles northwest of Letcher, and north of that place forms a rough strip about 2 miles in breadth nearly to James River. East of that stream and north of James Creek its main portion turns northeast. Its course northeast of Artesian is so faintly developed that it has not been traced. This member seems to have formed the main barrier which produced a shallow lake east of Woonsocket. From its southern side this lake had three or four outlets through the moraine: Long Lake, northwest of Letcher; a similar channel east of Letcher; and James River.

The Third or Antelope moraine lies along the west side of the area west of Wolsey in the form of a conspicuous ridge. It turns southeast and later to the east north of the main course of Sand Creek. It crosses James River on the north line of Sanborn County, becomes less prominent, and gradually curves in a northeast direction, becoming still less prominent, but is faintly developed west of Iroquois, where it has a north course, which continues beyond the limits of the

area. This may be considered the first member of the Third moraine and the most conspicuous part of it.

Lying approximately parallel with the first member are two or three lines of scattered knolls, beginning near the northern boundary, west of Broadlands, and curving around south of Huron. One of these is developed into a ridge 30 or 40 feet in height south of Cavour. As these have much of their course in a plain which probably was at one time a shallow lake, they are very feebly developed.

Along the inner side of the first member of the moraine, on the west side of James River, lies Cain Creek and east of it Pearl Creek. The portion of the moraine lying between Cain Creek and Sand Creek is traversed by two or three well-defined channels, reaching through the moraine from the valley of one stream to the other.

ANCIENT CHANNELS AND TERRACES.

Under this head are included all water-laid deposits of any considerable thickness except such as are in the immediate flood plains of present streams. In some of the shallow channels the older deposits may not be clearly separable from those of recent origin. In such cases the later are included under this head. The geologic map shows the location of these channels. They correspond generally with the present waterways, for the latter are the puny successors of the former, and yet the direction of drainage has been so changed in a few cases that some of the present valleys are connected by a network of older channels.

General character.—These channels vary from broad, shallow depressions, approaching lakes in character, through which streams passed for a comparatively short time, with a sluggish current, to troughs 20 to 40 feet in depth, in which the abundance of coarse material shows that they were occupied by vigorous streams for some time. In both cases the coarser deposit is usually sufficiently covered with finer material to afford rich soil. Where the channel deposits have been cut through by the deeper trenching of the later streams similar differences in the character of the deposits also occur. In some cases the old channel deposit is at a height of 50 or 60 feet above the present stream. In many cases, however, the old deposits have been only slightly eroded, because the later drainage has passed off in another direction.

We find the older channels changed to terraces, particularly along James River, where sometimes two, at different altitudes, appear. East of Huron, for example, these are found at about 40 and 60 feet, and east of Mitchell at about 80 and over 100 feet above the stream. They are not always distinctly marked, but may merge one into the other. The usual indication of such a terrace is a sharp, stony edge capping the river bluff and a generally flat surface extending for many rods.

FRAZIER ARTESIAN WELL, 10 MILES NORTHEAST OF MITCHELL, S. DAK., 295 FEET DEEP

Origin.—These ancient channels were developed during the presence of the glacier, and served to carry off the water from the front of the ice sheet in its different stages. The arrangement of the channels is evidence of the former existence of an ice sheet over this region. The size, and particularly the course, of some of the channels and the amount of coarse material found in them can not well be explained in any other way.

The order in which these channels were occupied may be learned from the map, where they are numbered; but it should be remembered that it is impossible to represent the order of their occupation with minute accuracy. In some cases the same channel has been used in opposite directions, and this change has perhaps occurred more than once. It will be noticed that the numbers are generally wide apart, but are sometimes found near each other, especially along the course of James River. In such cases it is intended to indicate that the channel had been used at times corresponding to the occupation of channels bearing those numbers and that the altitude of the formation does not clearly distinguish the order of time. Usually the later terraces are lower than the older, but this is not always the case. During the rapid unloading of débris from the ice the streams might at one time erode and at another time fill, so that waters are conceived to have been at nearly the same level at widely different times.

To give an example of the way in which the channels record the presence and different stages of the ice sheet and at the same time the varying directions in which the water flowed, attention is called to the channels west and south of Mitchell. (See Pl. III.)

When the ice occupied and was building the outer member of the Gary moraine, the drainage about the western edge was discharged along a channel which follows the course of Firesteel Creek and a southern tributary, and which leads west of Mount Vernon; then through the main branch of Enemy Creek to the center of T. 102 N., R. 61 W., when it passed by another intermediate channel into the north branch of Twelvemile Creek, thence by another crosscut to the south branch of the same stream. Into this the water from the ice sheet discharged by several outlets through the moraine. Since the course of this stream was on a higher level than the land farther east a short distance within the edge of the ice sheet, as soon as the ice began to recede the waters at several points, particularly near the north line of Aurora County, turned eastward through the outlets and followed for a time the edge of the ice, while it formed the second member in a way analogous to that already sketched. When the ice receded to the third member a similar change took place by the main stream, following the present course of Firesteel Creek to a point northeast of Mount Vernon before turning south. The course of Firesteel Creek was not occupied as at present until the ice had withdrawn beyond the junction of this stream with James River.

ANCIENT LAKES.

In connection with the channels, certain extensive areas which may be called ancient lakes should be considered. The use of this term does not imply that they were occupied at one time by water, although this may have been the case. It is probable that as the ice receded over them from the south toward the north the southern portion was first occupied by more or less ponded water and filled by the accumulating sediment from the streams draining the adjacent ice sheet; as the ice uncovered one portion after another these areas were successively filled with sediment in the same way, until, on the recession of the ice, they became flat plains, covered with sand or clay, with points of the underlying till rising above it like islands, and shallow channels winding about irregularly upon it. In some cases shallow lakes may have continued for an indefinite period. The largest and best example of this sort lies between Woonsocket and Forestburg, and extends from the outer portion of the third moraine to the fourth member of the second moraine. It covers an area of perhaps 50 miles. Over this area the surface is poorly drained and covered either with fine clay, commonly called gumbo, or with sand which is sometimes gathered in dunes by the wind. The gumbo seems to have been formed by the settling of clay in comparatively quiet water. The sandy areas from their position overlying the gumbo and till seem to have been formed as deltas by Sand Creek and Redstone Creek.

A similar area, not so clearly defined, surrounds Huron, especially on the south and east. This does not present so much of the gumbo surface, and the sand is also less marked.

Between Letcher and Mitchell and extending eastward across James River is another flat area which presents more of the usual surface of the till and less of the sand and gumbo.

GEOLOGIC HISTORY.

The granite which underlies most of James River Valley is considered to be part of the very old surface which existed before life appeared upon the earth. Whether it may be, in some cases, part of the original substance of the globe or whether it was first laid down in an ocean and afterwards changed to a crystalline rock by heat and pressure has not been satisfactorily ascertained.

The Sioux quartzite, which is exposed in the region east of Mitchell, belongs to a later epoch, and certainly was deposited in the ocean, probably upon the margin of a land area which existed farther east in Minnesota and Wisconsin. From that land, the waves, ocean currents, and tides, which were probably more vigorous then than now, derived the sands and spread them out over the bottom, much as now takes place on the seashore. As the material accumulated there was a gradual sinking of the ocean bottom till a thickness of several hun-

ARTESIAN WELL ON SCHLUND FARM, DAVISON COUNTY, S. DAK., 390 FEET DEEP, 40-GALLON FLOW.

dred feet had been laid down. There is no clear evidence that there was any animal life in that ancient ocean.

In the succeeding age there was a disturbance which resulted in the eruption of molten rocks which may have reached the surface and penetrated the sand strata in both dikes and sheets. It was probably during this period that the sand became consolidated into the hard stone we find to-day. Some of the igneous rock which appeared at that time has been found at Hull, Iowa; near Corson, in Minnehaha County; in Hanson County, sec. 26, T. 104 N., R. 58 W., and at Yankton. Probably in early Cambrian time this region was above sea level, and then followed the long period of erosion which is mentioned above as the Paleozoic gap. It may have been that the first upheaval of this region was a few hundred or a few thousand feet, but more likely the rise was by several successive elevations. It is probable that during the erosion of Paleozoic time several hundred feet of rock were carried away, and that the buried ridge extending through the Mitchell region is but a small remnant of the original rock mass. The land surface at this time may have been clothed with vegetation and occupied by animals of various kinds. During the last of the Paleozoic and the early part of the Mesozoic the seashore must have been beyond Missouri River on the west.

At the beginning of Jurassic time the land began to subside and the sea gradually advanced in central South Dakota, but apparently in this region a land surface continued until much of the Cretaceous period had passed, for the first deposits appear to have been the sediments of Dakota time. These were mainly sands deposited on beaches and in estuaries, but in intervals of quieter and deeper waters clays were also laid down. It is likely that the sands were carried by vigorous tidal currents and probably came in part from the disintegration of the quartzite along the adjacent shore. The clay may be traced with considerable confidence to the soil and fine material which were being washed from the land as the waters continued to advance toward the east.

At the end of the Dakota epoch the ocean waters overspread the region as far as southeastern Minnesota and the deposition of the Benton shales began. There were some short periods of shallow waters with strong currents, which deposited local layers of sand, but clays were the predominant sediments. In Niobrara time the waters were deep and clear in the greater part of the area, and great deposits of carbonate of lime accumulated, now represented by the chalkstone. At this time there was abundant life in the waters, including fish, huge reptiles, and mollusks. Deep waters and clay deposition continued during Pierre time, and probably several hundred feet of Pierre sediments extended across southeastern South Dakota. In the latter part of the Cretaceous there were at first shallow ocean waters of Fox Hills time, and then brackish and fresh waters in which the

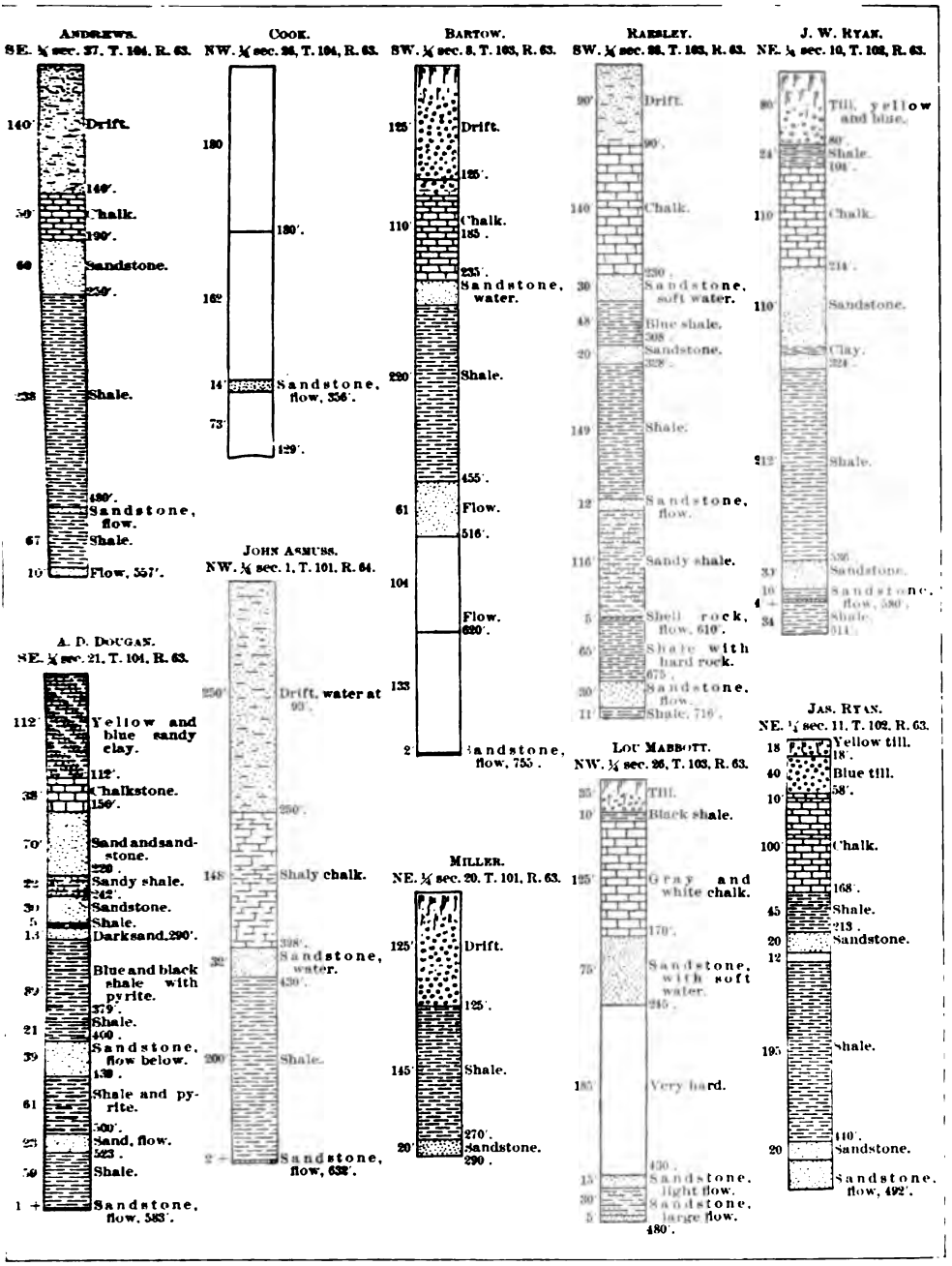
Laramie sandstones were laid down; but as these formations are absent to the southeast there is no evidence as to the conditions in southeastern South Dakota in this epoch, and presumably it was then a land surface. Probably the land surface continued during the Tertiary period, and some of the streams of later Tertiary time spread local deposits of sands in some portions of the region, but it is impossible to affirm what the conditions were in this area, because subsequent erosion has removed all such deposits. During the later part of the Tertiary period there was doubtless a large stream somewhere near the present position of James River, flowing toward the south.

The remarkable area furnishing artesian waters from underneath the till near Artesian may be considered as some evidence that the former course of James River lay along that line. The prominent range of knolls east of Mitchell suggests the presence underneath of a prominent point in the underlying rock which may have existed on the west side of the pre-Glacial James River. The lower land 3 or 4 miles east of the James, south of Alexandria, continuing to Hutchinson County toward the mouth of Wolf Creek, and the apparent recency of the trough of James River west of it below Rockport, as is indicated by the prominence of sandstone cliffs at several points, also the absence of chalk in a strip passing Farmer, Hanson County, combine to at least suggest that the pre-Glacial course of James River lay east of its present course through the southern part of this area. This suggestion has not, however, been tested by a comparison of significant well borings.

Into James River probably came White River, through the valley of Red Lake, White Lake, and Firesteel Creek. Those rivers doubtless had many small tributaries which had rapidly cut down the soft materials composing the surface. The elevated region in the southwestern part of Davison County may be considered a remnant of the old divide south of it.

Such were the drainage conditions until the Ice Age began, when it is supposed the climate became moister and colder. During the earlier stages of the Ice Age the ice had not broken over the divide between James River and Red River, and hence the streams, although swollen by rains, did not receive water directly from the ice. If the ice reached the boundaries of the State at any point, it probably did so in Minnehaha County, coming over from the Minnesota Valley, and Big Sioux and Vermilion rivers carried off the water.

During the Wisconsin stage the ice finally extended over the divide east of James River near the north boundary of the State, and steadily progressed down the valley until it had filled it to a depth in the center of 1,000 to 2,000 feet. At this time the ice extended as far west as Kimball, southwest of Lake Andes, south to Yankton, east to the East Vermilion, and southeast to Canton and Vermilion. During this stage the region was being ground down, chalkstone and softer



LOGS OF WELLS IN EASTERN AURORA COUNTY, S. DAK.

deposits were carried away, and the surface of the quartzite was planed and grooved as is found southwest of Bridgewater and at numerous other exposures.

This condition probably continued for hundreds of years, but in due time, possibly from a sinking of the northeastern part of the continent or from some general climatic change, the strength of the ice current was checked, and it gradually melted back, possibly until a portion of this region was uncovered. It is barely possible that the marsh deposits before referred to, found near the southwest angle of this area, were laid down at that time; but this does not seem likely, because they seem rather to be Cretaceous clays than till underneath them.

Again the ice sheet advanced and formed the first member of the Gary moraine. At that time the southwest angle of this area was uncovered, and also the northeast angle, and the drainage from the west side of the ice sheet passed through the valley west of Mount Vernon, and that from the east side through the valley east of De Smet. The southern point of the ice must have extended into the northeastern part of Yankton County. When the ice receded to form the fourth member of that moraine, the west branch of Sand Creek drained down the channel west of Pony Hills west of Woonsocket, while the drainage from the east side was largely by Rock Creek.

While the Third or Antelope moraine was forming, the North Fork of Sand Creek, fed by several streams from the ice sheet, washed much débris into the basin east of Woonsocket. At the same time Redstone Creek sustained a similar relation on the east side of the James.

The last appearance of the ice in this area was an almost stationary glacier. The ice probably lay in extensive blocks in the basins east of Huron, even after James River was running near its present course, a little northwest, perhaps with icy banks. A portion of the basin north of Huron seems to have been more open and to have received much sand and silt from the melting ice sheet farther north, not only through the James, but by a channel which led past the present location of Broadland.

Afterwards the streams assumed their present courses, and, though somewhat larger than at present, had little effect upon the surface of the country except to deepen the channels which were occupied by permanent water. It is believed that James River had cut nearly to its present depth before the ice had disappeared from the State. The main geologic development of the country since the disappearance of the ice has been in the formation of the soil, which has gone on by the accumulation of alluvium along the principal streams. To this may be added the deepening of fine material over the general surface by the burrowing of animals, the wash from the hillsides, the settling of

dust from the atmosphere, and the accumulation of the remains of vegetation.

ECONOMIC GEOLOGY.

There are no deposits of mineral ores or of coal in this area. Fragments of coal are sometimes found in the drift, but they have been brought by the ice or by streams from the northern part of James River Valley.

Valuable quarries of red quartzite have been opened near Spencer and Alexandria and quarries might be operated at perhaps a dozen other points. Their location may be learned from the geologic map (Pl. III) by noticing the outcrops of Algonkian, or quartzite. Chalkstone and the upper Benton sandstone have also been used for the construction of buildings, but the material is not considered sufficiently durable. Much of the stone locally used for foundations and other rough buildings is derived from the drift. It consists of granite, limestone, and greenstone bowlders, which are very durable and with a little care may be neatly laid.

Deposits of clay of economic value are not certainly known to exist. A bed was opened southeast of Mitchell, from which brick were manufactured at that place, but the frequent occurrence of lime nodules in it rendered the preparation of it too expensive for practical purposes. It is not improbable that diligent search would find beds of silt near James River, or of gumbo in the lake basins, in sufficient quantity to be of some local value in making brick, but there is apt to be so much lime and coarse material mingled with it that brickmaking will not be extensively carried on in this region.

Sand and gravel deposits may be found very generally in the more important old channels.

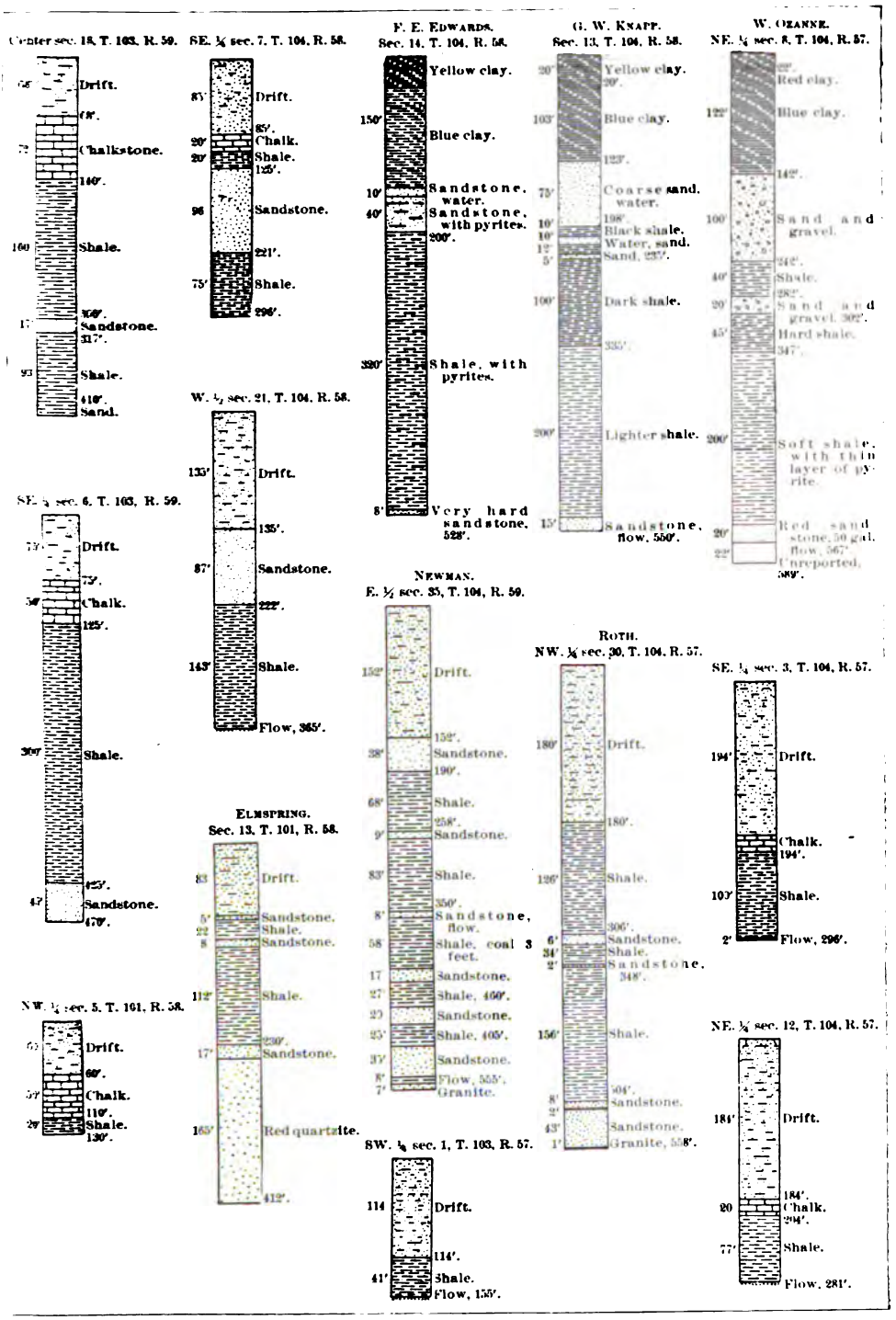
WATER SUPPLY.

This resource is not only of prime importance, but considerable knowledge of its distribution, character, and accessibility is directly obtained by geologic investigation. Its occurrence may be classified first into surface waters and subterranean waters. The former will include lakes, streams, and springs, and the latter wells—both pump wells and artesian wells.

Surface Waters.

LAKES.

The numerous basins found upon the surface of the till form important natural reservoirs for holding the rainfall, and doubtless assist much in diminishing the evil effects of droughts. Only the largest of them retain water from year to year. Only a few of those marked upon the map (Pl. III) have areas of open water. More frequently they are filled with grass or reeds. Some of them have become convenient reservoirs for the overflow of artesian wells.



LOGS OF WELLS IN HANSON COUNTY, S. DAK.

Some of the largest cover an area of 2 or 3 square miles, as a lake 5 or 6 miles northwest of De Smet.

STREAMS.

Mention of these has already been made under the heading "Geography." Attention is here called to an interesting feature which is usually manifest in the upper and middle courses of the smaller streams. Most of them show the occurrence at irregular intervals of deep pools, sometimes 5 or 6 feet deep and a rod wide, and extending in length for several rods. These are separated by deposits of sand and gravel, which, in the latter part of the season, are often dry and grassed over. These pools usually contain water, which is kept comparatively pure by a slow circulation which passes down from pond to pond through the gravel bars. In time of rain these bars are overflowed and the decomposing vegetation in the ponds is apt to be swept out.

SPRINGS.

The pond holes just mentioned might be named in this connection, as they are supplied from ground water which drains in below the surface. But there are also clearer examples of springs, and the springs of this region are supplied from three more or less distinct sources.

One source is the sand or gravel capping the terraces. Springs of this sort abound near James River in the ravines south of Rockport and for a few miles north and south of Forestburg and on the Firesteel north of Mitchell. Another source, scarcely separable from the first, is from sand strata considerably below the surface. Several fine springs of this class are found along the lower course of Pearl Creek and along the east side of James River, between Huron and the mouth of Shue Creek. These all furnish clear, cool, hard water, which deposits in some cases considerable travertine or limestone.

One spring, at least, is known to come from the Benton shale. It is found on the west side of Twelvemile Creek, about 2 miles north of Hutchinson County. It is known that the chalkstone is often saturated with water, but at no point in this area are circumstances such that it is brought to the surface in the form of springs.

A third source of springs, producing some of the finest to be found anywhere, is the Benton sandstone. All that are known from this source probably spring from the first sandstone below the chalk. It is possible, however, that near the quartzite leaks from deeper strata may reach the surface. One of the most notable of these springs is found about 3 miles east of Ethan. It has been impossible to measure the quantity of water escaping here because of its discharge through a mass of moss covering an acre or more; but some idea of the amount of water discharged may be obtained from the fact that

after cold weather, with a temperature of 20° below zero, it kept open a pond several square rods in extent and sustained a stream a quarter of a mile long, free from ice in the middle. In the lower course of Enemy Creek, extending 2 or 3 miles above its mouth, are several large springs supposed to be fed from this source. It is possible, however, that some of them may be supplied from water high up in the same stream through crevices in the quartzite. Another fine spring, one of the best examples of this class, is found in the northeast corner of sec. 27, T. 104 N., R. 60 W., on the east side of James River. It is a circular basin about 30 feet across, in the flood plain of James River. Its depth has not been determined, but it keeps flowing a stream 4 or 5 feet wide and 3 to 6 inches deep.

Subterranean Waters.

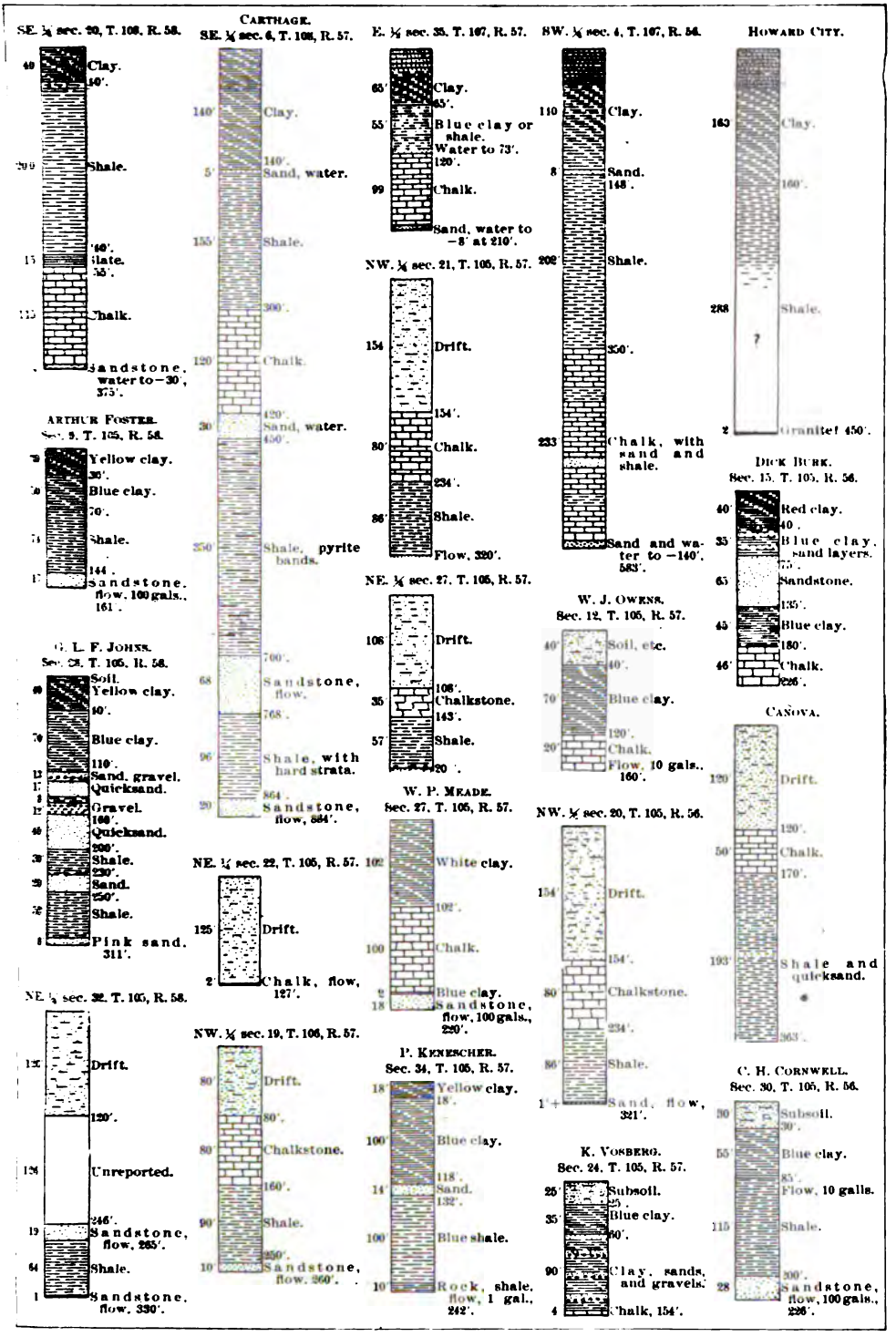
WELLS.

Reference has been made already in the discussion of surface waters to the close connection between water holes along watercourses and the motion of waters near the surface in old channels and in the upper part of the till. Thus far natural waters only have been discussed. We will now consider those obtained artificially. Wells may be conveniently classified into shallow wells, pump wells, and artesian wells.

SHALLOW WELLS.

This term includes wells supplied from waters which have recently fallen upon the surface and which can be obtained without penetrating any impervious layer. Such wells are easily obtained closely adjacent to any of the present watercourses, whether these contain any standing water upon the surface or not. They may be found also frequently in the vicinity of basins, especially after a wet season. Such wells in this area may obtain water at a depth of 10 to 30 feet, but do not afford a copious or permanent supply except when located near the bottom of a large depression or near a channel draining a considerable area. The reason for this is obvious, since the water comes from the rainfall only and the region is subject to long-continued droughts. Of this class only those can be counted upon as a permanent supply which are so situated as to draw from a large catchment basin. In digging such wells, if no water is reached before the blue bowlder clay is struck, none will be found until the clay is passed through.

As regards the area under consideration, it may be suggested by what has already been stated that extensive areas where shallow wells are permanent may be found in connection with the flat areas about Huron and Woonsocket. In the sand of the Woonsocket basin copious wells are found in the eastern and southern part, west of Sand Creek, at a depth of 10 to 20 feet. So, also, north and northwest of



LOGS OF WELLS IN MINER COUNTY, S. DAK.

Huron, in the vicinity of the broad, meandering channels, abundant water is found, except in very dry seasons, by digging 10 or 15 feet.

In the area southwest of Huron, where the stratum of sand is found about 80 feet below the surface, permanent wells occur, which may be considered as belonging to this class. They are from 30 to 35 feet deep.

PUMP WELLS.

Under this head we include the deep wells in which a tubular or force pump is often necessary, or where the water supply is reached only after passing through an impervious layer. Three distinct geologic horizons supply such water in this area, particularly in the northern part—viz, (1) the sands below the till; (2) either a porous stratum or crevices, probably in the chalk, and (3), of most importance, the sandstone below the chalk, which is the first regular water-bearing stratum of the Benton-Dakota series.

As before explained, there is usually below the till a stratum of sand or gravel. This is also commonly filled with water. As soon, therefore, as the till has been drilled through at ordinary level, the water rises several feet, sometimes very nearly to the surface. The water from this source is heavily charged with lime, and sometimes with iron. It is commonly cool and wholesome. In some regions, however, either at first or soon after the well is dug, the water becomes so impregnated with the soluble salts which abound in the bowlder clay that it becomes offensive and sometimes injurious. This has been especially true of the wells in the plains west and north of Woonsocket. It has not been ascertained whether this is due to the abundance of mineral salts accumulated in this basin before the deposition of the till or to organic matter deposited there, or whether it is due to the water standing in the till after the well is dug. Similar cases are reported occasionally from other places.

A few localities of limited area have failed to furnish water from this horizon. It may be supposed that they occur where the original surface of Pierre and Benton clay was so elevated that it was not submerged by the waters attending the advance of the ice sheet, so that the till was deposited in close contact with the Cretaceous clay. Such a locality as this is reported from sec. 24, T. 110 N., R. 64 W. Another is near the southwest corner of sec. 31, T. 112 N., R. 63 W., and doubtless others occur, but they are not so frequently reported since more desirable horizons have been discovered, and this one is less relied upon.

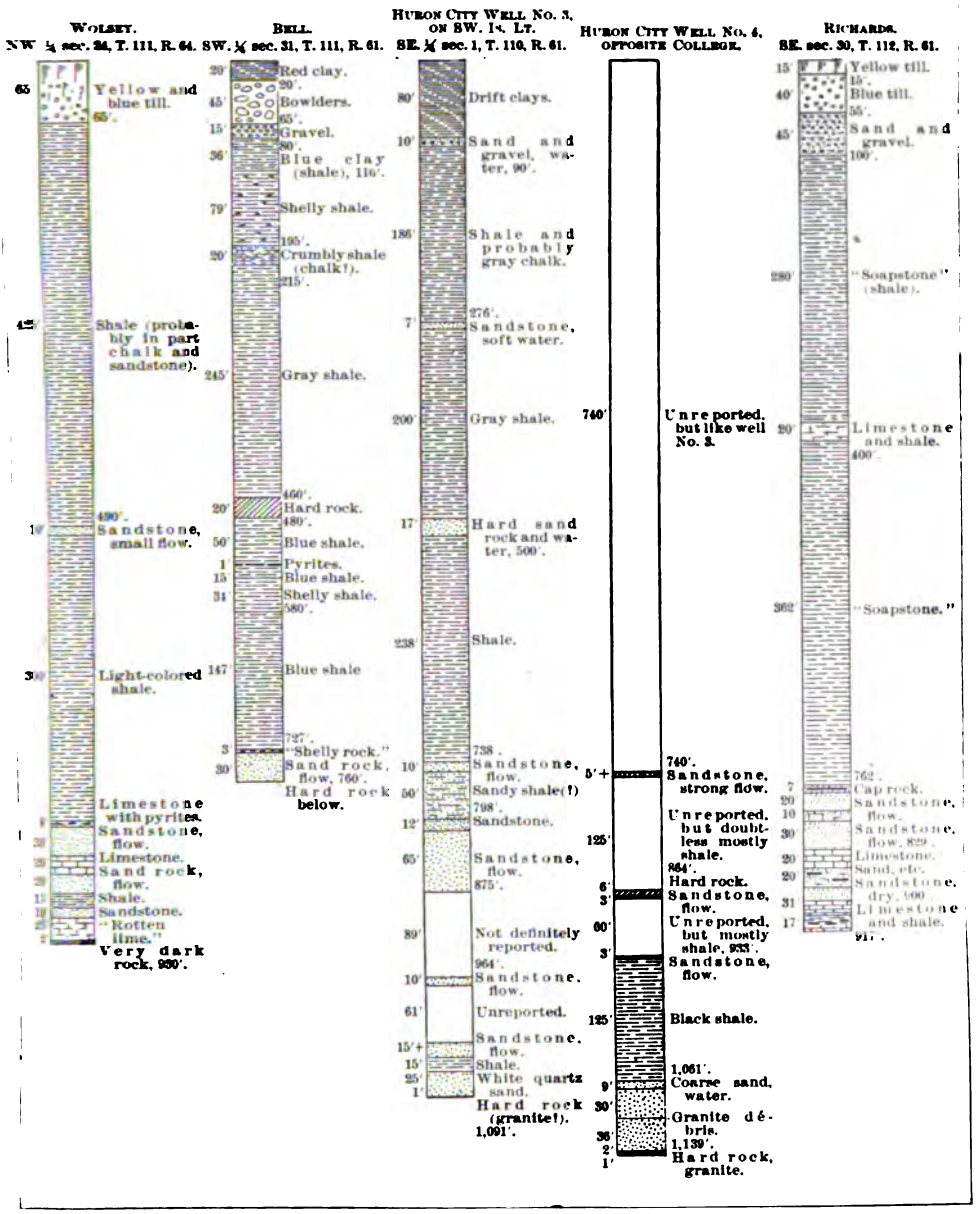
Similar failure to find water occurs over the chalkstone, where it lies so high that the water has drained away to the surface, as southwest and south of Mitchell. On the other hand, in certain areas the water in this horizon is under such pressure that it produces valuable flowing wells. Such an area begins in the southern part of Floyd Township (T. 108 N., R. 60 W.) and extends southeast past Artesian,

which place derives its name from the early discovery of this fact. Here the flowing water is obtained by simply penetrating the till. In Plano Township (T. 104 N., R. 59 W.), also water is obtained from beneath the till, but it is probably derived from the upper Benton sandstone beneath.

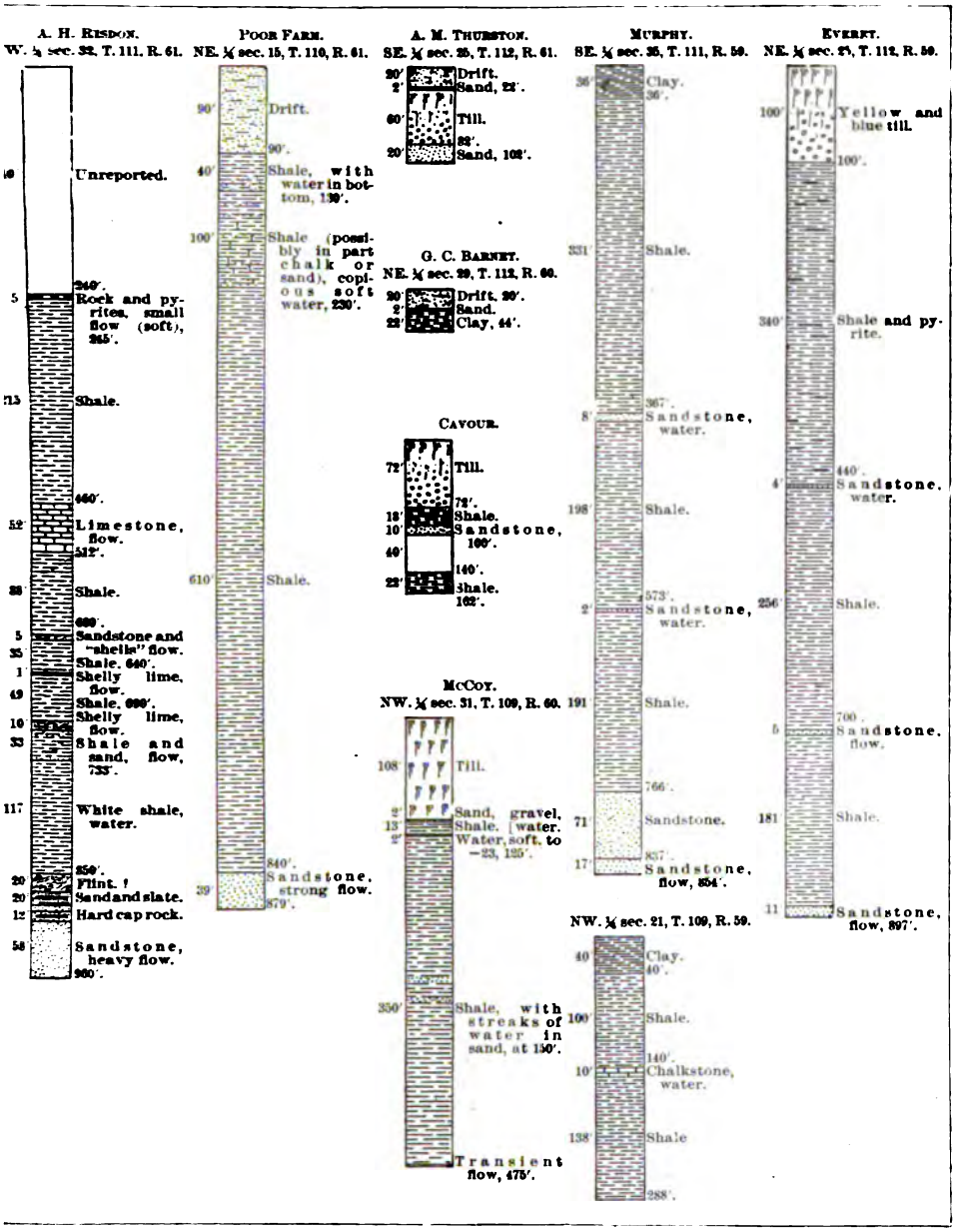
In the northern part of the area another horizon is much relied upon for furnishing pump wells. It seems to lie in the shales above or beyond the chalkstone, but this has not been clearly proved. East and southeast of Huron it lies at a depth of 115 to 175 feet, the depth increasing toward the north, but it seems to lack the regularity of most water strata. Some of the shallower soft-water wells about Plankinton are from this horizon rather than from the sandstone a little below. These wells may be due to crevices in the shale or chalkstone which supply water from more extensive horizons lower down. The water from this source is commonly spoken of as from the "soapstone," and is soft.

The third and most important pump-well horizon is in the widespread sandstone at or near the top of the Benton formation, which throughout this area seems to lie just below the chalk, but south of the latitude of Mitchell a mass of clay or shale is found between the chalk and the water stratum. In some localities this sandstone furnishes flowing wells, particularly in the trough of James River, and along the valley of Firesteel, Enemy, and possibly Twelvemile Creek, except where the altitude of the land is too high for the water to come to the surface. Here again there is an irregularity, especially in the northern part of Sanborn County, for the waters seem in some cases to have escaped upward into the chalkstone, so that in many localities water appears as soon as the chalkstone is reached. Some well drillers have the idea that the chalkstone alternates with the sandstone in the same horizon, and where the chalkstone is present the sandstone is either absent or reduced to an insignificant thickness. This opinion has gained some strength from the observed relation of the chalkstone to the sandstone where it is exposed along Firesteel Creek; but it is probable that the sandstone extends underneath the whole, but with unequal thickness, and that the chalkstone lies in the depressions of its upper surface, and wherever it is not too much permeated with clay it becomes a medium of water distribution from the sandstone, which is the main course of the water.

The waters from this horizon are universally, so far as known, soft—i. e., the mineral salts dissolved are those of soda, which do not interfere with the use of soap. The water is also more palatable than that from the lower horizons of the Benton or Dakota. It supplies what are known as "soft-water wells" throughout this area, except in the more elevated portions, where it lies so deep that few have sunk wells to it. Moreover, the pressure in it would only be sufficient to



LOGS OF WELLS IN WESTERN BEADLE COUNTY, S. DAK.



LOGS OF WELLS IN EASTERN BEADLE COUNTY, S. DAK.

raise the water to from 200 to 300 feet below the surface in the elevated region around De Smet. Furthermore, in the southeastern part of this area, which is underlain by quartzite near the surface, this water horizon has not been recognized, probably owing to the fact that the stratum is absent, or is so confused with the sand below the till that it is not distinguished from the higher horizons underneath the drift.

It has been already noted that the same horizon furnishes flowing wells in one locality and valuable pump wells in another. It follows from this that some of the horizons, which are discussed more fully under artesian wells, are really a source of pump wells in the more elevated portions of the area. For example, the main stratum furnishing artesian water in James River Valley may furnish an unfailing source for pumping beyond the limits of the artesian area indicated upon the map.

ARTESIAN WELLS.

There have been considered water horizons as deep as the first one below the chalkstone, which is referred to the top of the Benton formation. Attention has been called also to the fact that this and the higher strata furnish artesian wells in certain narrow limits. In the further discussion under this head mention is made particularly of water strata lower down having such pressure that they are the source of artesian wells over extensive portions of this area, in fact, over most of the area.

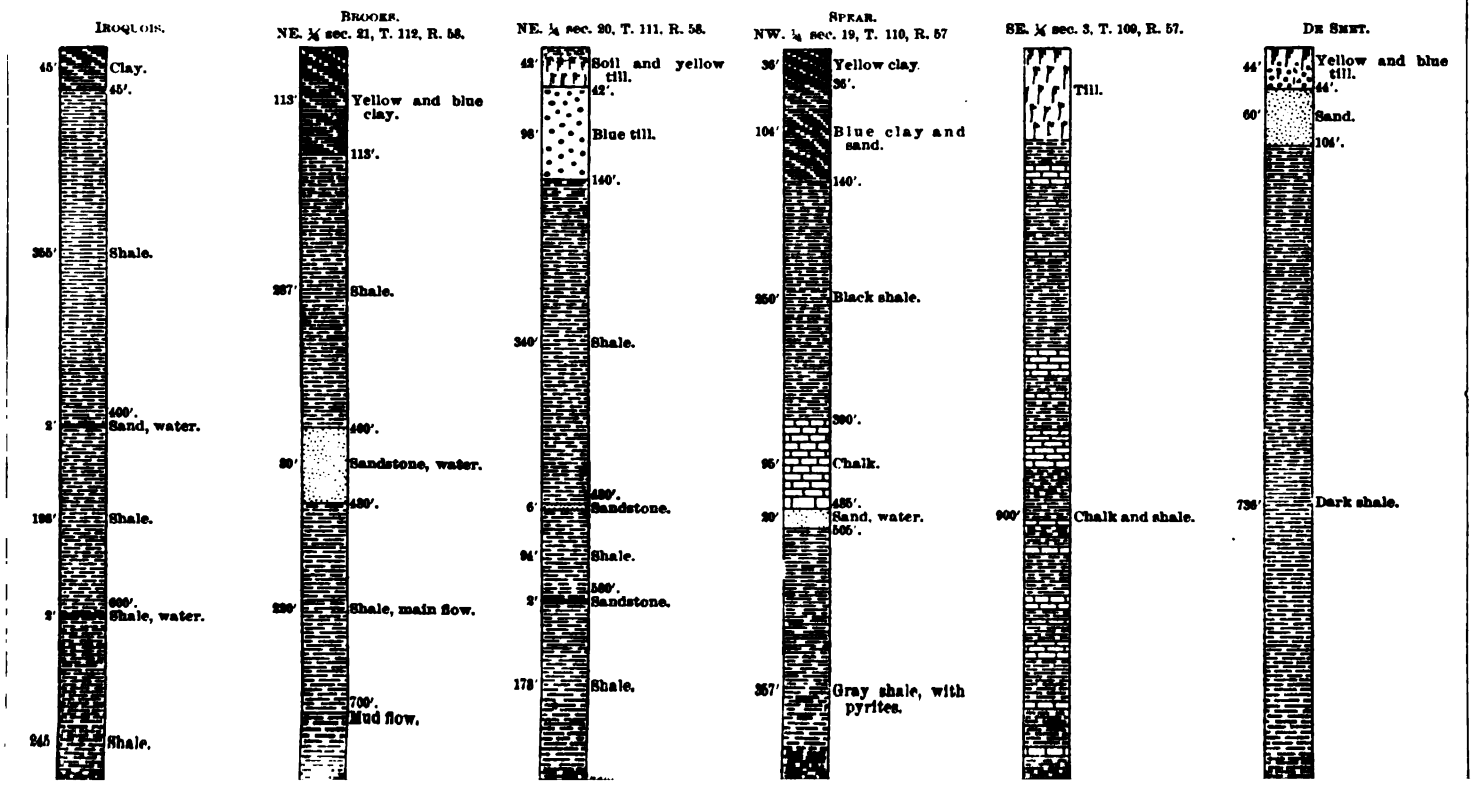
The character of the Dakota formation has been discussed already, but perhaps the fact has not been emphasized sufficiently that the different strata of sand often are independent of one another. This is mainly indicated by the usually well-marked difference in pressure. For example, in the same well, one stratum may sustain a closed pressure of 75 pounds to the inch and another below it, separated from it by 15 or 20 feet of shale, or sometimes even by a less thickness of hard rock, may furnish a pressure of more than 100 pounds. Moreover, there are differences in the character of the water of different strata.

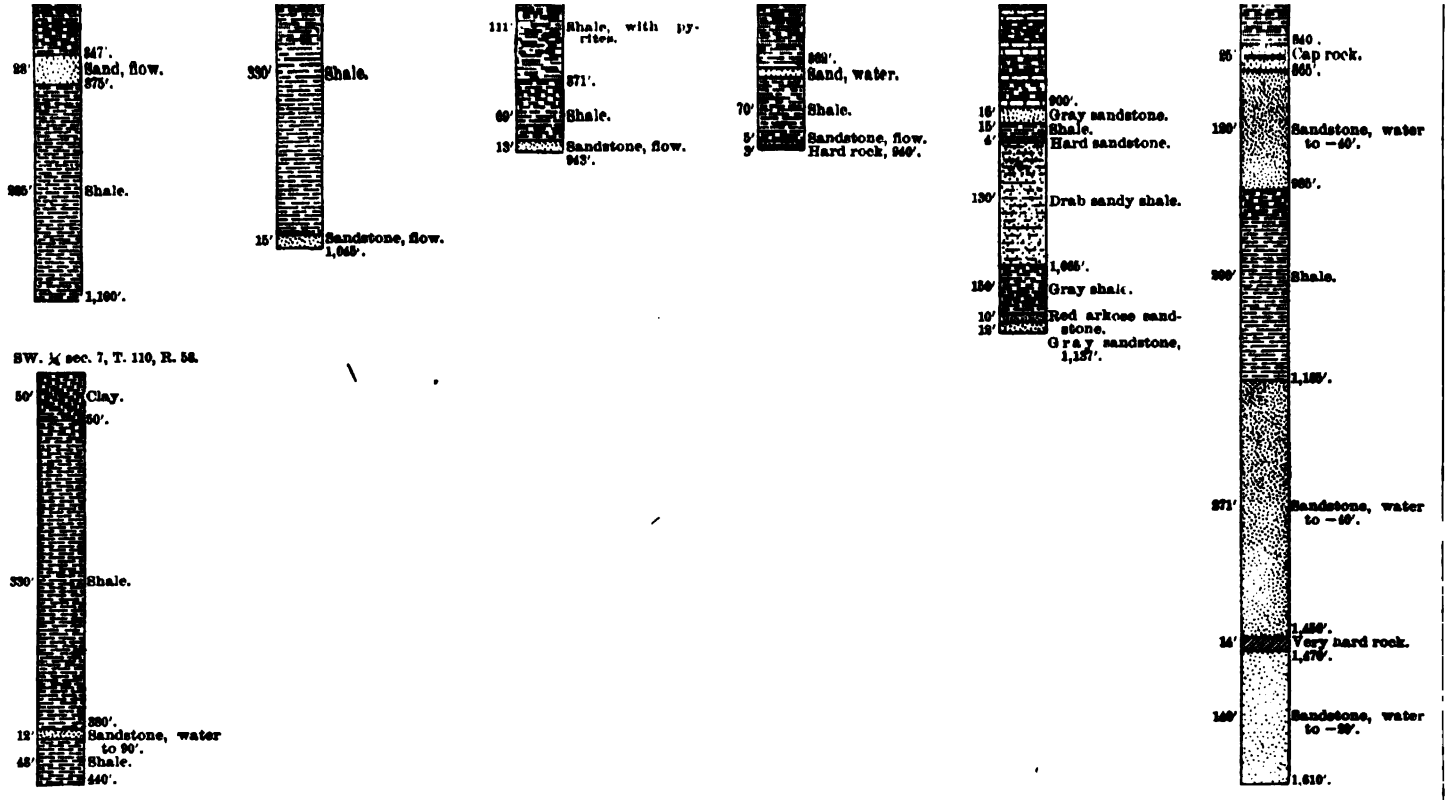
The second sand stratum in the Benton formation furnishes numerous wells from 250 to 500 feet deep in western Hanson County and Davison County, and the same horizon may be traced throughout the northern portion of this area, but it becomes so thin and the water furnished so meager that it is usually overlooked or cased off. Its depth is about 500 feet at Huron, 600 near the southeast corner of Beadle County, about 350 feet in the region about T. 101 N., R. 64 W., although it is now usually neglected except in Davison and Hanson counties, where its supply is more copious. It nevertheless has good pressure, that may in years to come afford a supply when some of the strata which are now more drawn upon have become exhausted.

The next horizon is the Dakota sandstone, which is mainly relied upon for the more copious wells throughout the whole area. The larger wells are nearly all supplied from this horizon. In order to obtain the largest amount of water, it is customary to penetrate deep into this formation, so as to gain the combined flow from several sandstones. The water is usually hard, but suitable for all farm and household purposes. It is also used for supplying cities, not only for domestic purposes but for fire protection. In some cases the pressure is sufficient for fire protection, for instance, at Huron and Woonsocket. The depth at which this horizon is struck, of course, depends in part upon the altitude of the surface and in part on the depth of the beds. The stratum lies higher in the vicinity of the quartzite ridge and lower toward the north, so that it is reached at about 750 feet at Huron, at about 650 at Woonsocket, at about 850 at Iroquois, at about 600 at Artesian, at about 900 near Manchester, at about 450 at Mitchell, and 546 at Plankinton. The depths to the principal flow are shown on Pl. I.

Lower horizons of Dakota sandstone supply the Melville, Riverside, Wolsey, Plankinton, and T. O. Storla wells, and the lowest sandstones supply some wells in the vicinity of Huron, as the Risdon and the new city well, which reach a stratum lying immediately above the bed rock. The extent of the lowest stratum has not been widely explored. It is absent in the southeastern portion of the region, contiguous to the high underground ridge of quartzite.

Pressure.—In general the pressure increases downward in the different flows. This is what might be expected, mainly because of less chance for leakage from these beds along the eastern margin of the basin. The lower water-bearing strata may be supposed to reach to a higher altitude along the intake zone in the Rocky Mountains and Black Hills, which would give a higher initial head, but so slight a difference would be lost in the great general loss of head eastward. The lower beds also have the first chance to become filled by the surface waters, for they lie farthest back on the mountain slopes. The pressures of the wells over much of this area have not been satisfactorily obtained. The reasons for this are several. The first pressures, or those of the first wells opened, were much higher than any that can be obtained at present, at least without closing all other wells in the vicinity, which is impracticable. For example, at Huron the pressure first reported, and this is believed to have been from the first main flow, was 120 pounds, but at present from none of the wells of the city can a pressure of more than 70 pounds be obtained. Another difficulty is that many of the wells have been finished in such a way that it is impracticable to apply a gage at the present time. On account of corrosion of the pipes the older wells can not safely be closed to obtain pressure. Another source of error comes from imperfect casing in the well, so that it is impossible really to


Iroquois.
45' Clay.
45'.
355' Shale.
400'.
2' Sand, water.
198' Shale.
600'.
2' Shale, water.
245 Shale.
Brooks.
NE. ¼ sec. 21, T. 112, R. 58.
113' Yellow and blue clay.
113'.
287' Shale.
400'.
20' Sandstone, water.
420'.
220' Shale, main flow.
700'.
Mud flow.
NE. ¼ sec. 20, T. 111, R. 58.
42' Soil and yellow till.
42'.
98' Blue till.
140'.
340' Shale.
480'.
6' Sandstone.
Shale.
500'.
2' Sandstone.
178' Shale.
Spear.
NW. ¼ sec. 19, T. 110, R. 57
36' Yellow clay.
36'.
104' Blue clay and sand.
140'.
250' Black shale.
390'.
95' Chalk.
485'.
20' Sand, water.
505'.
357' Gray shale, with pyrites.
SE. ¼ sec. 3, T. 109, R. 57.
Till.
900' Chalk and shale.
De Smet.
44' Yellow and blue till.
44'.
60' Sand.
104'.
738' Dark shale.



LOGS OF DEEP WELLS IN KINGSBURY COUNTY, S. DAK.

close the well because of underground leakage. This applies either in taking the closed pressure with a gage or the pressure by allowing the water to rise in the standpipe. On the artesian map (Pl. I) the pressures, where determined, have been recorded by stating the number of feet above sea level to which the water would rise. There are also drawn upon the map contour lines which bring out graphically the slope of pressure or hydraulic gradient of the first main flow or the third water stratum below the chalk. This will be found not to agree with all of the closed pressures recorded, for some of them are from other horizons.

From a study of these contours it would be seen that the pressure diminishes toward the margin of the artesian area in all cases, but with a complexity which is difficult to express in language. For example, the slope of pressure in the southwestern part is toward the northeast. In the northern part of Hanson County it is toward the south. It will be noticed also that although the pressure diminishes mainly toward the east from Huron eastward there is a slight increase of pressure. This may at first be thought difficult of explanation, but when we remember that several large wells have been flowing in James River Valley, supplied from this source, for several years, we may explain the apparent anomaly by considering an extension of this local exhaustion so as to affect the area farther east. Moreover, since in the highlands, away from the larger streams, we may suppose the leakage from the different water-bearing strata is much less than in James River Valley, and that the region east of that stream may be supplied from the north, we have further light upon the subject.

Limits of the artesian area.—The limit of the artesian area, as drawn on the artesian map, is estimated from the closed pressure observed in the few wells near it, and as these are lacking toward the south the limit there is less reliable. Moreover, the pressure mapped by contours and used in making the estimate is that of the first main flow or the third water stratum below the chalk. It is probable that from lower strata, which very possibly underlie Miner County, a somewhat higher pressure may be obtained. If so, the limit will be correspondingly shifted toward the east. It is not probable, however, that the area will be much increased from this cause. The pressure of the Risdon well was 165 pounds when the city well of Huron was 120. That difference would be equivalent to an altitude of 104 feet, but the difference of pressure would doubtless be much less near the margin.

It should be remembered that the limit in this area from the north line to the south line of Miner County is due to lack of pressure, not to the absence underneath of the water-bearing strata. Hence, deep pump wells may draw from the artesian supply eastward to an indefinite distance. In the counties farther south, however, the water strata, excepting one or two of the upper ones, are lacking. Moreover, the latter merge into the sand under the till, so that the waters are quite different in character.

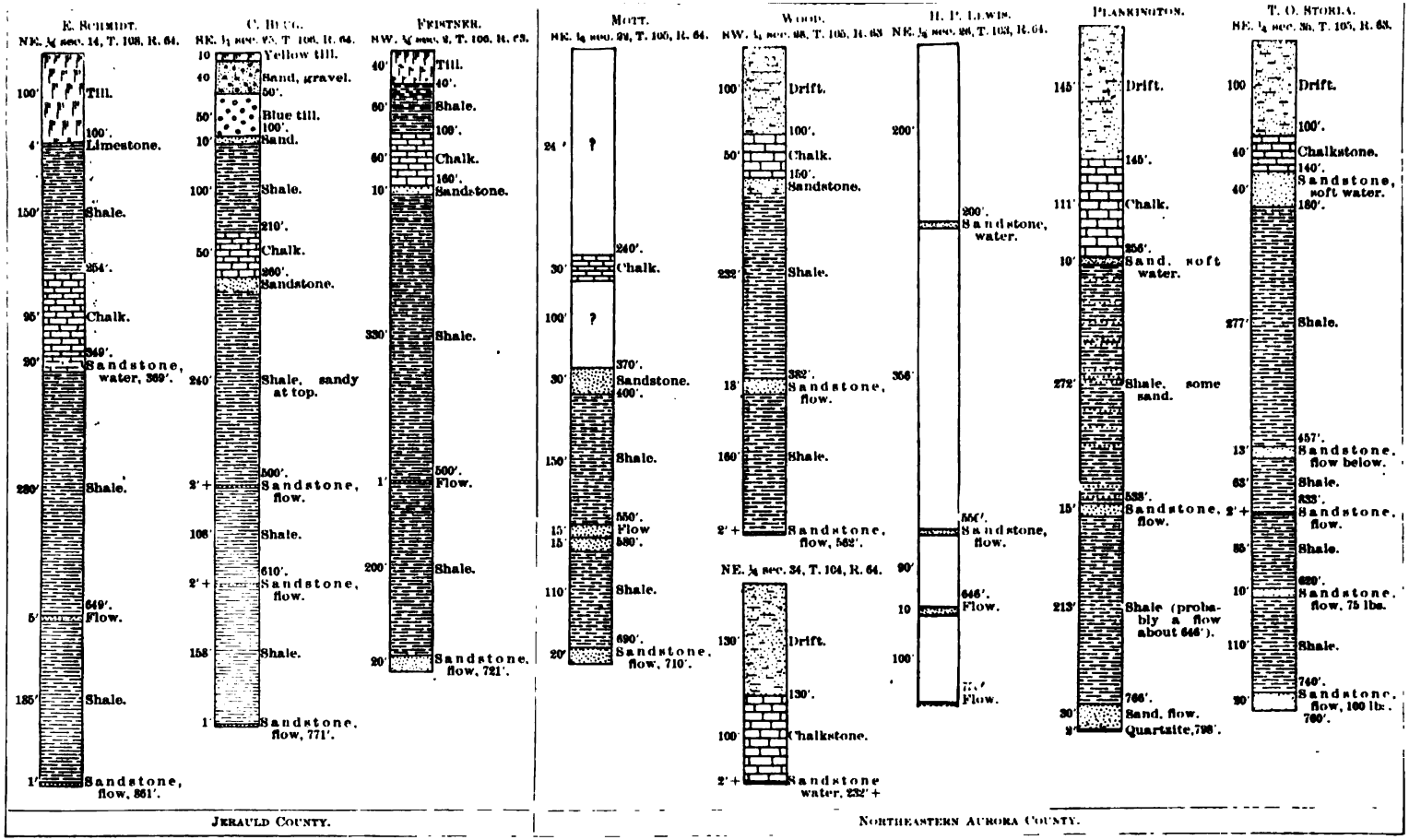
Causes of apparent decline in pressure.—It is now generally admitted not only that the copiousness of the flow of wells diminishes, but also that their closed pressure declines. This becomes evident without the use of instruments; first, the distance to which the water is thrown from a horizontal pipe grows shorter; then, after a time, the stream, which first filled the pipe, fails to completely fill it. In some cases a test with a gage shows merely a decline in copiousness without material decline in pressure. It may be accounted for by the deposition of mineral matter about the bottom of the pipe in such a way as to clog the pores of the sand through which the water comes, but more commonly the pressure has been found really to diminish. Thus in the wells at Huron some that once showed a pressure of 120 pounds when closed now fail to reach 80. Similar facts have been reported from Mitchell, Mount Vernon, and Plankinton.

The unwelcome conclusion impressed by these facts has led many to search for reasons other than the one first suggested, viz., the partial exhaustion of the artesian supply. It is claimed, and apparently substantiated by facts, that new wells frequently have a pressure equal to that of early wells supplied from the same source. Since the closed pressures, however, are less frequently taken than formerly, and from the nature of the case liberal margins are sometimes made for leakage, it is difficult to prove the strict truth of this statement.

The first cause of apparent decline is a less copious flow, occurring either, at first, when a new well taps the same water-bearing sandstone, or by the clogging of a well that once flowed freely. The former case may result from the fine character of material or the thinness of the water-bearing stratum where penetrated; but the more significant examples are those which may result from the clogging of the well. As wells are usually finished by extending the pipe into the water-bearing rock and perforating it as high up as that formation extends, it will be readily seen that the surface opened for the delivery of water to the well is equal to the whole perforated portion of the pipe. As the water continues to flow, sand will gradually accumulate on the inside of the pipe, and so gradually diminish the surface supplying water to the well. Something of the same sort may less frequently occur even when the pipe terminates in the cap rock. As time passes, sand gradually works in from the sides, and portions of the cap rock are undermined and dropped down, so that in such cases the free access of the water is considerably checked.

Theoretically the closed pressure would be the same whether the well is flowing freely or not so long as the head of the water is the same. If the well becomes clogged, as suggested above, the only difference in pressure would be that when a gage is attached it takes longer to reach a maximum point. As this rise may be very gradual, some errors of reading have doubtless resulted because the observers did not wait long enough.

Another cause of diminished pressure is leakage. This may be



LOGS OF WELLS IN JERAULD AND AURORA COUNTIES, SOUTH DAKOTA.

either visible from the imperfect closing of the pipe or it may be invisible below the surface of the ground. As is well known, pipes deteriorate rapidly under the influence of most artesian waters, and it becomes almost impossible to close the joints perfectly. Where there is a considerable extent of piping, as in the distributing pipes of a city, all joints are not certain to be stopped. Hence this may explain sometimes the apparently diminished pressure in older wells.

The diminished pressure in a particular well may sometimes result from the opening of another well not far away. The distance to which this influence may extend will, of course, be greater where the water-bearing stratum is of coarse texture, and therefore the usual supply of water more free.

In cases where water has been drawn freely from several wells, or even from one large well, there is no doubt a local depression in the head or lowering of pressure, which will take considerable time to restore. This might occur and yet cause no permanent decline of supply.

Notwithstanding all the considerations offered thus far, it seems not unlikely that the rapid multiplication of wells in a region may really reduce the pressure over the whole region to the amount of a few pounds. It is therefore important that facts should be collected and sifted to ascertain whether this is the case, and if so, the amount of diminution. In view of such a possibility of overtaxing the supply it would seem desirable to limit in some way the number of large wells allowed to flow freely. A single 1,000-gallon a minute well would be sufficient to supply 450 wells furnishing 100 barrels a day, which would be an adequate supply for an ordinary farm.

Closing of wells.—Much damage is sometimes done by the free running of wells. In some cases large wells have been drilled with the intention of irrigating, and sufficient rainfall for a series of years has rendered that unnecessary. The water from such wells has been allowed to run to waste, thereby drawing unnecessarily upon the general supply. Moreover, it has often rendered considerable land in the vicinity unproductive. The practice, therefore, of closing wells when not needed should be recommended. The only objection to this is the possibility that wells when closed will become clogged. This danger may be avoided by the gradual closing of a well, even when it is known to carry some sediment. When the water runs clear, and especially where the well has never thrown sand, there is very little danger. Some large wells made to furnish power are habitually kept closed when not in use without serious injury. In case a well does become clogged by the settling of sand it may often be opened by letting down an iron rod and churning it up and down until the flow is started.

To avoid producing too sudden changes in the flow, which may produce injurious effects at the bottom of the well, the opening and closing should be done gradually.

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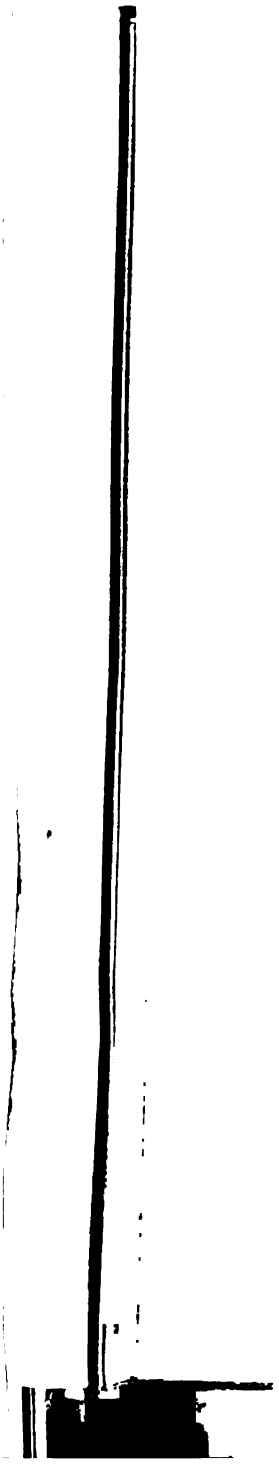

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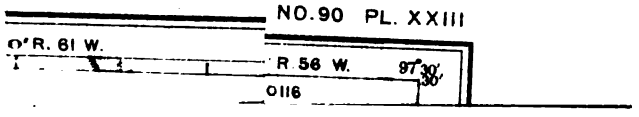
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DEPARTMENT OF THE INTERIOR

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

THE NATURAL FEATURES AND ECONOMIC DEVELOPMENT

OF THE

USKY, MAUMEE, MUSKINGUM, AND MIAMI DRAINAGE AREAS IN OHIO

BY

BENJAMIN H. FLYNN

AND

MARGARET S. FLYNN

WASHINGTON
GOVERNMENT PRINTING OFFICE
1904

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
Washington, D. C., October 8, 1903.

SIR: I have the honor to transmit herewith a manuscript for a Water-Supply paper treating of the four principal drainage areas of the State of Ohio and describing the economic development that has taken place within each.

The greater part of this manuscript was written by the late Benjamin H. Flynn, who for some years prior to his death was employed by the United States Geological Survey as field assistant in charge of stream measurements in the State of Ohio. After the death of Mr. Flynn the manuscript was completed by his widow, Margaret S. Flynn, who for several years had taken an active part in his professional work, especially in connection with his duties as engineer for the Ohio State board of health.

This paper treats of the natural features and development of the areas drained by the Maumee, Sandusky, Miami, and Muskingum rivers and contains valuable data with reference to the flow of these streams and the natural advantages they offer with reference to water power and domestic water supply. Of particular interest is that part of the paper which deals with the early history of the water powers, their general relation to the canal systems of Ohio, and the decline in value that has taken place since the establishment of modern methods of manufacturing, trade, and transportation.

The chapter dealing with public water supplies comprises unusually complete information with reference to the sources, equipment, and value of the systems that were installed during the latter part of the last century.

This paper is one of a proposed series dealing with hydrographic matters in the several States of the Union and showing the advantages that the various river systems offer and the manner in which they have been developed, from an economic standpoint.

Very respectfully,

F. H. NEWELL,
Chief Engineer.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

THE NATURAL FEATURES AND ECONOMIC DEVELOPMENT OF THE SANDUSKY, MAUMEE, MUSKINGUM, AND MIAMI DRAINAGE AREAS IN OHIO.

By BENJAMIN H. FLYNN and MARGARET S. FLYNN.

INTRODUCTION.

The data for the following paper were gathered during what might be called a sanitary survey of the drainage areas of Ohio rivers, made for the State board of health. In the original reports of this work an attempt was made to give complete information regarding the sanitary condition of these areas, but little was said concerning the hydrography except in connection with public water supplies. This statement applies especially to the Sandusky and Maumee basins, as they were taken up first, and data concerning the water resources could receive but little attention, owing to the extra time required to plan the work. In connection with the work in the Muskingum and Miami basins, during the next two years, more attention was paid to the gathering of this information, though it was, of necessity, subordinate to the sanitary survey.

The Sandusky and Maumee drainage areas were inspected during the latter half of 1898, the Muskingum in 1899, and the Miamis in 1900. In the summer of 1903 letters were sent out to various cities and villages asking for information in regard to certain statistics which are subject to variation. From the mass of details in the original reports and from these letters there have been assembled fairly complete data concerning the hydrography of these areas, so far as this is given, and the more salient facts concerning their sanitary condition.

GENERAL DESCRIPTION OF DRAINAGE AREAS.

Four separate drainage areas are described in this report—the Sandusky, the Maumee, the Muskingum, and the Great and the Little Miami.[a] Their general locations are shown in fig. 1.

LOCATION.

The Sandusky basin is situated in the central part of the northern half of the State, and its waters drain to Sandusky Bay, an arm of Lake Erie. The Maumee basin occupies the northwest corner of

[a] For general purposes the Miami and Little Miami drainage basins may be treated as one area.

Ohio and small portions of the adjacent States of Indiana and Michigan, and its waters empty into Maumee Bay, at the western end of Lake Erie. The Muskingum basin occupies a large section of the eastern part of the State south of the Lake Erie drainage divide, and its run-off is to Ohio River. The Miami basins occupy a large portion of the southwestern part of Ohio and a considerable area of the southeastern part of Indiana. The Little Miami basin lies east and south

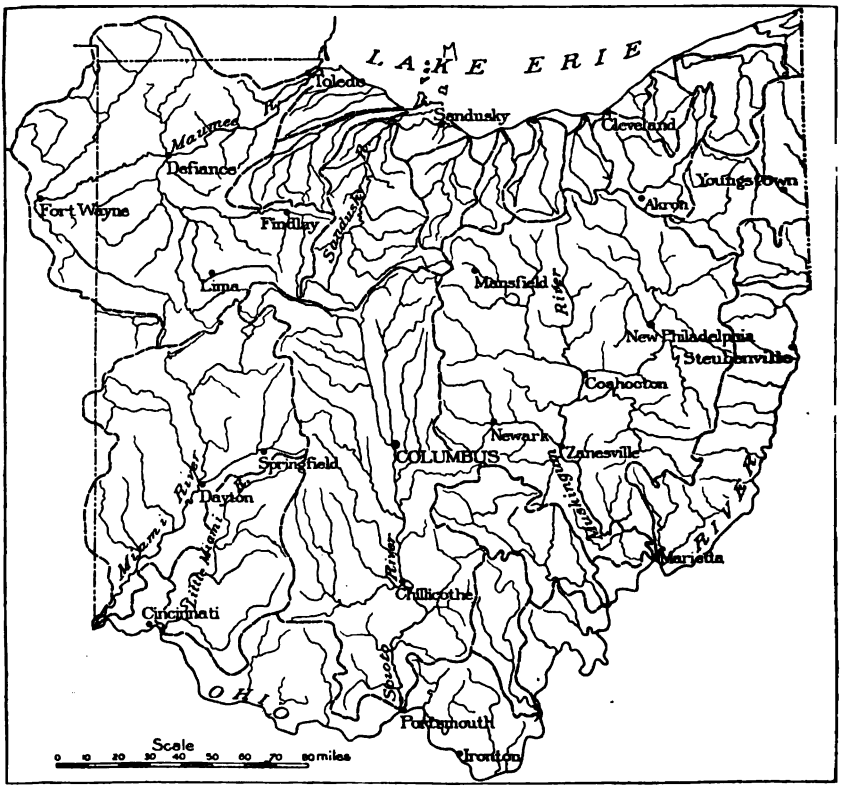

FIG. 1.—Map of Ohio, showing drainage basins.

of that of the Great Miami and west of the basin of Scioto River. The waters from its southern border are carried by short, turbulent creeks directly to the Ohio.

AREA.

In point of size the Muskingum ranks first, with an area of 7,797 square miles, nearly one-fifth of the area of the State. The Maumee is next in size, with an area of 6,344 square miles, of which 4,702 square miles are in Ohio, 1,303 in Indiana, and 339 in Michigan. The total area of the Great Miami basin is 5,247 square miles, of which 1,355 square miles are in Indiana and the remainder in Ohio. The Little Miami basin has an area of 1,709 square miles, all of which is

in Ohio. The Sandusky basin has an area of 1,581 square miles, being almost exactly one-fourth as large as the Maumee. These areas were computed with a planimeter from the best available map of the State and are fairly accurate.

POPULATION.

According to the Twelfth Census, the population of the Sandusky drainage area is 126,591, of which 49,370 may be considered as urban and the remainder, 77,221, as rural. In explanation of these terms, it may be said that all cities or villages with an estimated population of 1,000 or more are classed as urban, while those below this figure are included with the rural population. This low figure is assumed in order to bring into the urban classes all the towns that have public water supplies or sewerage systems and to keep in the rural class all the villages not so provided.

On a similar basis of classification the Maumee basin is estimated to have a total population of 640,128, of which 339,268 are classed as urban and the remainder, 300,860, as rural. Of the above total population, Ohio has 478,078; Indiana, 141,899, and Michigan, 20,151. The total population of the Great Miami basin is 520,698, of which 258,859 are classed as urban and 261,839 as rural. Of the total population, 89,814 are in Indiana, divided as follows: Urban, 32,062, and rural, 57,752. The total population of the Little Miami basin is 120,959, of which 39,645 are urban and 81,314 rural. The Muskingum basin has a total population of 612,784, of which 229,794 are urban and 382,990 rural.

The following statement shows the population per square mile of each district:

Population per square mile of drainage basins of Ohio streams.

	Rural.	Urban.	Total.
Sandusky	49	31	80
Maumee	47	53	100
Great Miami	53.6	59.5	113.1
Little Miami	47.6	23.2	70.8
Muskingum	49	29	78
Average	49	39	88

In the small area of 216 square miles lying between the two Miami basins and tributary to Mill Creek and to Ohio River there is a total population of 375,571, of which 350,894 are urban and 24,677 rural. This gives an average population per square mile of 1,739. A large part of this area is occupied by the city of Cincinnati, which accounts

for the unusually large urban population, amounting to 1,625 per square mile, against 114 per square mile of rural.

The slight excess in the average of the rural population of the Muskingum basin is due principally to the large number of small mining settlements scattered over the coal regions of that basin. Its agricultural population in much of its area is probably below the average, but the latter is brought up by the mining communities.

TOPOGRAPHY AND SOILS.

In general topography the Sandusky and Maumee are much the same. Both lie in glaciated areas and have the level plains and rolling elevations characteristic of such districts. The Sandusky basin is generally more rolling than the Maumee, especially the Ohio portion of that area. The large central area of the Maumee basin is taken up by a fertile, level plain, while the outer edges of this fan-shaped district are more hilly, especially to the north and west.

The Sandusky drainage area consists of ordinary agricultural land interspersed with small patches of timber. There is practically no waste land in the whole district, except a small marshy area around the mouth of the stream. Corn and wheat form the staple crops, with the usual admixture of hay and oats.

In the Maumee basin agriculture is not so far advanced and there are large tracts of land still covered with brush and shrubbery, but little good timber remaining. The extensive hard-wood forests of this region were a profitable field for the operation of stave and heading factories, and as a result of the extent to which these industries were carried on there is now much waste land here. In the cleared areas of the central and southern portions of this area corn is the staple and almost the only crop, but farther north the dairy and stock-raising interests are large and much land is kept in pasture, with a good per cent under cultivation for feed.

With reference to surface configuration, the Miami basins occupy a place between the Maumee and Sandusky basins on one hand and the Muskingum on the other. In this district there are greater extremes of elevation than in any other portion of the State, but the slopes are more uniform and the surface is more rolling than in the Muskingum basin, and there is none of the level prairie that is found in the northwestern part of Ohio.

The northwestern part of the Great Miami basin and the northeastern part of the Little Miami consist of comparatively level elevated areas, marked, however, with rolling glacial moraines. Between these two districts and connecting the extremes of elevation—425 feet above sea level at the confluence of the Great Miami and Ohio rivers and 1,540 feet just east of Bellefontaine—there is a wide strip of hilly and heavily rolling land with a rather steep slope to the southwest. A narrow strip of high land running lengthwise through this area forms the dividing line between the Great and Little Miami basins.

As the entire area of both basins has been glaciated, the surface presents the smoothed-over appearance that characterizes such districts, and no abrupt changes are to be found except where the streams have cut out their valleys in the soft drift.

The surface is covered with wide areas of heavy limestone soil, crossed by narrow stretches of loamy and gravelly soil. Some of the stream valleys contain extensive alluvial deposits, which form very fertile areas. Much of the clay subsoil of the southern part of the Little Miami drainage area is covered with loam, forming on the level uplands excellent farming districts.

As the whole district is essentially an agricultural one, the timber was rapidly cleared off and but small isolated wood lots now remain over the tillable portion. Along the steep hillsides and the streams there are still found stretches of forest, much of which is scrub or young growth, all the merchantable timber having been cut out.

The lower and rougher portions of the Little Miami basin contain the most heavily wooded areas to be found in either basin. Here, on the steep hills bordering the streams, there are still to be found many good tracts of timber, the inaccessibility of which has so far prevented their destruction.

Corn, wheat, and tobacco are the staple crops, the last being important in both districts. In and around Cincinnati extensive dairy interests have been built up. Good roads cover nearly all of both basins, indicating better than anything else that they are rich districts, inhabited by a prosperous and enlightened people.

The Muskingum drainage area is entirely different from the others under discussion. The terminal moraine passes diagonally across this district from northeast to southwest, leaving the northern and western part glaciated and the large central and southern portion unglaciated. The glaciated region is level in but few portions, but the rising, rolling ground makes an excellent agricultural district. The soil is here composed of glacial drift, and is usually deep and fertile. It is especially adapted to the raising of wheat, and large crops of this cereal are harvested annually. In the lowlands corn is also a flourishing crop. At one time, it might be added, these districts supported a large amount of stock, but the decline in prices has practically done away with grazing, especially with the raising of sheep for wool. It is to be supposed that the present increase in price will reestablish this industry.

The eastern, central, and southern parts of the Muskingum drainage area, or the unglaciated portions, are very hilly and rough, and the streams run in deep valleys cut in the bed rock. Here the soil is of a poorer quality, as it is formed by the disintegration of the native rock, and not as good a field is offered for agriculture except in the stream valleys, where there are long strips of alluvial deposit, which yield enormous crops of corn. The unglaciated section also corre-

sponds roughly to the mining district, a fact which has tended to decrease the agricultural value of much of the land.

The fertile fields of the northwestern part of the area and the hills of the southeastern part have been stripped of timber, the former to make room for agriculture and the latter for use in the mines; and as a result nearly all the timber left standing is to be found between the agricultural and mining sections.

GEOLOGY.

The Sandusky and Maumee basins lie, respectively, east and west of the Cincinnati axis or anticline, so that in the Sandusky basin the formations occur successively in long, narrow strips running nearly north and south. Starting at the east, the ages are as follows: Carboniferous, including only the Mississippian limestone and the Waverly; Devonian, including the Ohio shale, the Black shale, the Hamilton, and the upper Helderberg; finally, Silurian, with only the lower Helderberg, Rondout waterlime, Niagara, Clinton, and Medina appearing. In the Maumee basin these appear in reverse order, but in less regular strips and trending to the northwest instead of due west.

In a few localities in both basins where the drift was thin it has been removed for stone quarrying, but the industry could not be called an important one.

As stated before, the Great and Little Miami areas have both come under the influence of the glacial action which covered the original strata with a layer of drift that varies in thickness from 5 to more than 100 feet. Practically all the bed rock belongs to the Silurian system; the Hudson River group, the Utica shale, and the Trenton limestone covering the entire southern portion of both basins, while to the north and east there is a wide band of Niagara, Clinton, and Medina, and to the extreme northeast is found the lower Helderberg and Rondout waterlime.

On the high elevation east of Bellefontaine there appear small areas of both the Devonian groups, the Hamilton and upper Helderberg, and the shales. Near Dayton and in the valleys of the Stillwater at Covington and Mad River below Springfield there are extensive limestone quarries. At the latter place and at Cedarville, on Massicks Creek, a considerable quantity of limestone is taken out to be burned for lime.

In the Muskingum drainage area sedimentary deposits of only one age appear, the Carboniferous, with the following groups: Coal Measures, conglomerate, Mississippian limestone, and the Waverly. The first two of these occupy the large eastern and central section of the watershed, and the last two the western part. The Coal Measures contain many excellent veins of coal, which are extensively mined over a considerable area of this watershed. The above-named strata also furnish fire clay, limestone, and some building stone.

DRAINAGE.

SANDUSKY RIVER.

The Sandusky basin contains but one important drainage line, namely, the Sandusky River proper, which rises in the extreme eastern part of its drainage area and flows at first westward then northward to Sandusky Bay. Into this main stream there flow a large number of small creeks, among which are Broken Sword, Tymochtee, Sycamore, Honey, Wolf, Muskalong, and Green, the latter entering the river almost at its mouth.

Sandusky River is 115 miles long and has an average fall of 6.41 feet per mile. Its headwaters have an elevation of 1,282 feet above sea level, or 709 feet above Lake Erie. Near its headwaters the fall per mile is about 28 feet, but this is reduced to 7 feet between Bucyrus and Upper Sandusky, and thence the fall decreases gradually to 2.5 feet per mile below Fremont. Above Fremont the river has a steady flow, but below that town it becomes sluggish, winding in and out through low, flat, and in many places marshy land, until it reaches Sandusky Bay, where it spreads out over a broad plain covered with grass and lilies. Originally the headwaters of this river were situated at several springs 4½ miles east of Crestline, but since the water from these has been appropriated for the supply of that city little or none of the flow reaches the river. The headwater streams are now West Branch and "Stink" Creek, both of which are fed to a small extent by spring water, but to a much larger extent by the sewage of Crestline.

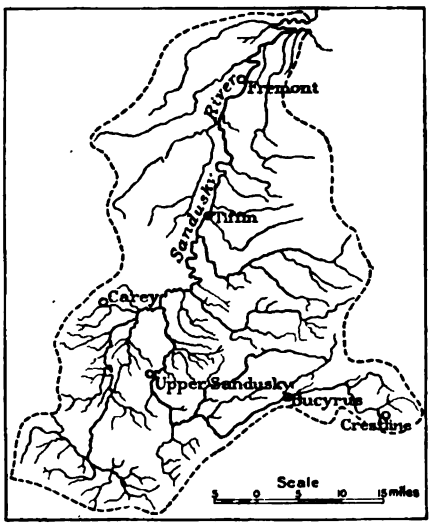

FIG. 2.—Map of drainage basin of Sandusky River.

MAUMEE RIVER.

Maumee River is formed by the junction of St. Joseph and St. Mary rivers at Fort Wayne, Ind. Its main drainage line starts at the extreme western edge of the drainage area and runs northeastward to the bay of Toledo. To this main line, Maumee River proper, there are four important tributaries, Tiffin and St. Joseph rivers on the north and Auglaize and St. Mary on the south. The Tiffin and Auglaize join the Maumee at Defiance, and a very symmetrical drainage basin

would be formed were it not for the fact that the Tiffin is much smaller than the Auglaize. The last-named stream has itself two large tributaries, Blanchard and Ottawa rivers, which enter it from the east.

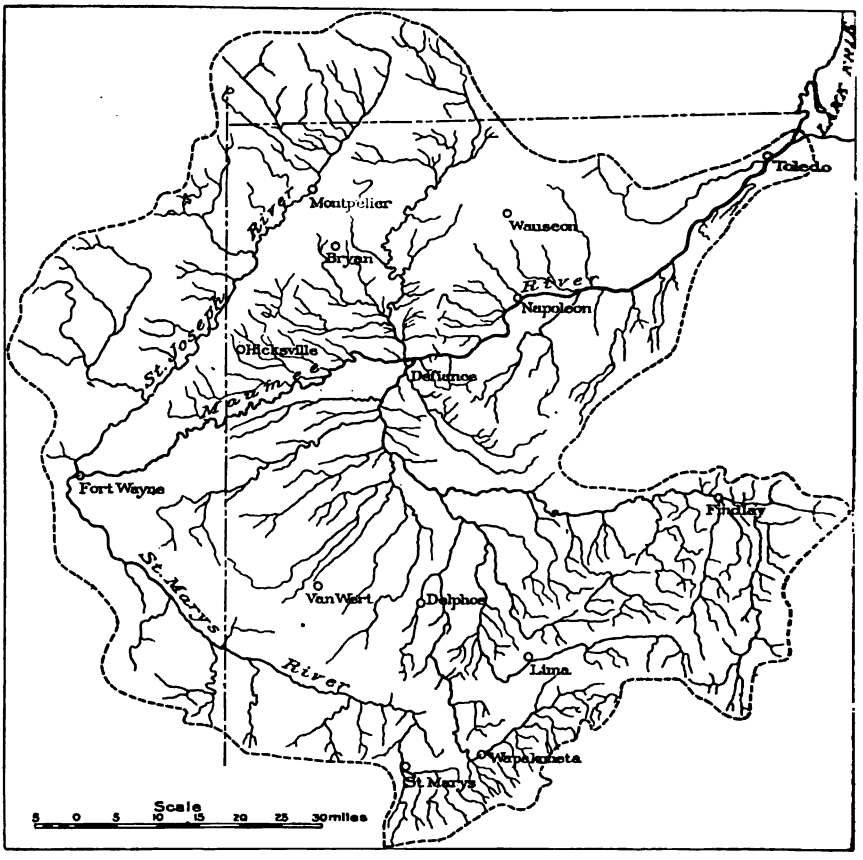

FIG. 3.—Map of drainage basin of Maumee River.

Length and fall of Maumee River and tributaries.

Stream.	Length in miles.	Total fall in feet.	Fall per mile in feet.
Maumee	150	164	1.1
Tiffin	65	293	4.5
St. Joseph	100	313	3.1
Auglaize	74	325	4.4
St. Mary	100	238	2.4

GREAT AND LITTLE MIAMI RIVERS.

The main drainage line of the Great Miami basin starts in its extreme northeastern part and runs southwestward, joining Ohio River at the Ohio-Indiana State line. Until Dayton is reached it occupies the central portion of the basin, but from that point down it lies very close to the eastern edge of its drainage area. Below Dayton its tributaries, not considering Whitewater River, consist of Indian, Sevenmile, and Twin creeks, each of which drains a long and narrow strip of territory. At Dayton the main stream receives two large tributaries—Mad River from the east and the Stillwater from the west.

In the Little Miami basin the main drainage line rises in the extreme northern part of the district and flows southwest, keeping close to the western boundary for its whole length, and finally joining the Ohio a few miles above Cincinnati. Its principal tributaries, all flowing in from the east, are the east branch of the Little Miami, Todds Fork, Cæsars Creek, and Massicks Creek, the first named being the most important.

The length, total fall, and fall per mile of the main streams and tributaries in both basins are shown in the following table:

Approximate gradient of Miami rivers and tributaries.

Stream.	From—	To—	Distance in miles.	Total fall in feet.	Fall per mile in feet.
Great Miami River	Ohio River	Hamilton	33	145	4.4
Do	Hamilton	Dayton	44	160	3.6
Do	Dayton	Piqua	33	110	3.3
Do	Piqua	Degraff	27	120	4.4
Do	Degraff	Headwaters	26	90	3.5
Do	Ohio River	do	163	625	3.8
Stillwater River	Dayton	Covington	30	150	5
Do	Covington	Headwaters Greenville Creek.	36	240	6.7
Do	do	Headwaters Stillwater River.	32	165	5
Do	Dayton	Headwaters Greenville Creek.	66	390	5.9
Do	do	Headwaters Stillwater River.	62	315	5.1
Mad River	do	Springfield	23	170	7.4
Do	Springfield	Headwaters	35	300	8.6
Do	Dayton	do	58	470	8.1
Twin Creek	Great Miami River	do	48	450	9.4
Sevenmile Creek	do	do	34	510	15
Little Miami River	Ohio River	Milford	13	66	5.1
Do	Milford	Waynesville	35	180	5.1
Do	Waynesville	Headwaters	46	450	9.8
Do	Ohio River	do	94	696	7.4
East branch Little Miami River.	Little Miami River	Batavia	12	105	8.8
Do	Batavia	Williamsburg	19	200	10.5
Do	Williamsburg	Headwaters	42	300	7.1
Do	Little Miami River	do	73	605	8.2

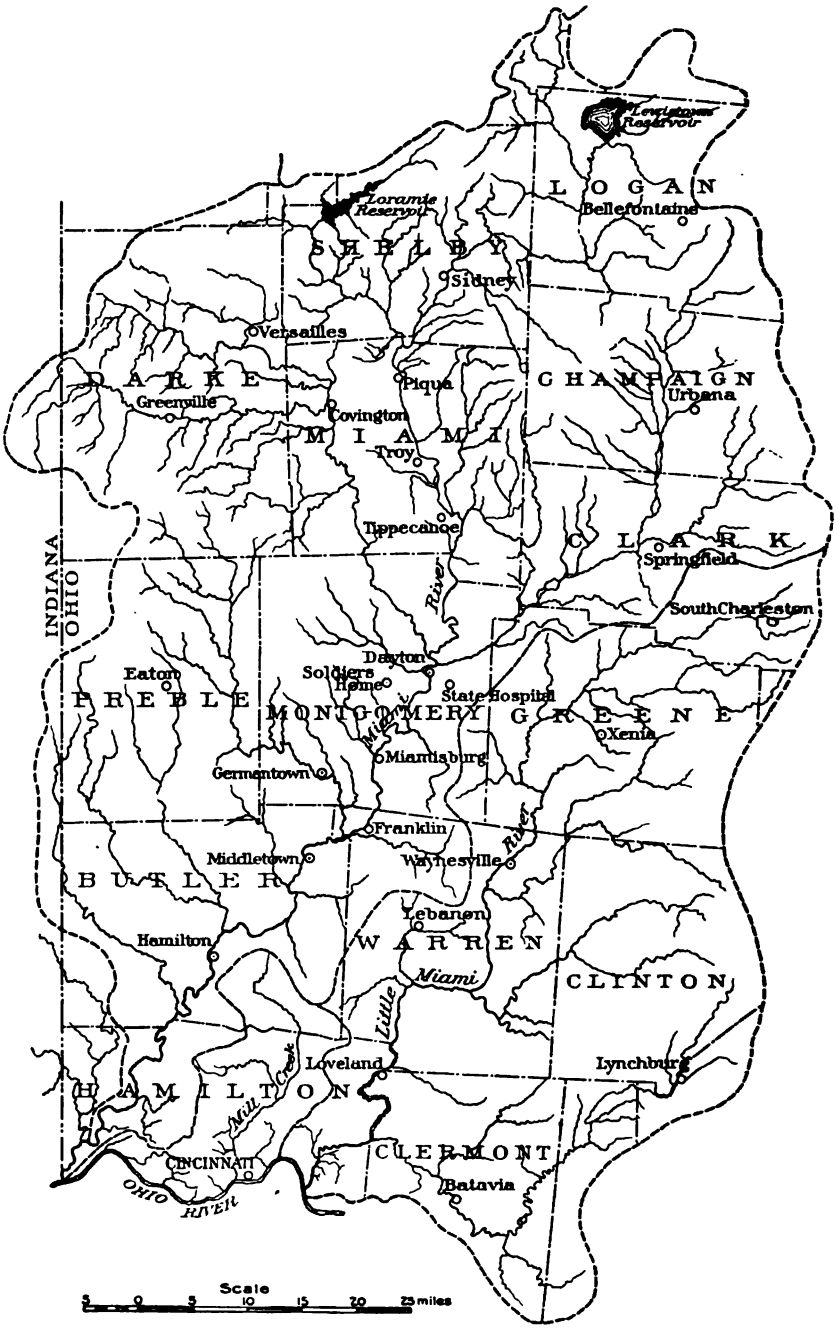

FIG. 4.—Map of drainage basins of Great and Little Miami rivers.

It is evident at once from this table that all the streams of the Miami basin flow at very rapid rates. Even the lowest average gradient, 3.3 feet per mile, found between Dayton and Piqua, is sufficient in a large stream to give a swift current. The gradient of the Great Miami is very uniform throughout its entire course, more so than is usual in so long a stream. Mad River also has a very uniform but a much steeper gradient. Sevenmile and Twin creeks, coming down from the high land near the Indiana line, have a very high gradient, the highest in either of the two basins. The upper part of Little Miami River and the east branch of the same stream also have high rates of flow, induced by steep gradients. At one time the excellent water powers on these streams were utilized by numerous mills, but now many of these have been abandoned. A discussion of available water powers is given later in connection with the descriptions of water powers still in use.

Owing to the general use of the land for agricultural purposes no extensive tracts of swamp land remain unreclaimed. Many large areas have been ditched and tiled, and much that was formerly waste land has been turned into exceptionally fertile fields.

The valley of Mad River between Dayton and Springfield, except immediately below the latter city, is low and swampy, much of it being so low that it can with difficulty be drained for use. Along this stream in Champaign County there are large areas of low, wet, peaty land which have not yet been successfully drained.

MUSKINGUM RIVER.

The main drainage system of the Muskingum basin consists of Muskingum River and its four principal tributaries, namely, Licking Creek, Wills Creek, Walhonding Creek, and Tuscarawas River, the two last-named streams uniting at Coshocton to form the Muskingum proper. The three streams designated creeks are large and could very properly be called rivers.

Tuscarawas River and Walhonding Creek drain the eastern and western parts of the upper portion of the drainage basin, and Wills Creek and Licking Creek, together with Muskingum River, drain the eastern, western, and central parts of the lower portion. Tuscarawas and Muskingum rivers could very properly be considered as one stream, forming the central drainage line of the watershed.

The pre-Glacial drainage of this area was along lines entirely different from the present ones. It is claimed that Muskingum River turned westward at Dresden and flowed through Licking Creek Valley to the Scioto. It is also believed that the Scioto flowed eastward through this same valley, then northward through the Muskingum, Tuscarawas, and Cuyahoga basins into Lake Erie. It seems to be established that the Muskingum did not have its present outlet. Some of the facts that have led to this opinion are, the traces of a

large channel through the Licking Valley, the small size and recent excavation of the present Muskingum River channel south of Zanesville, and the deep channel of the Tuscarawas to the north, where in many places the bed rock is lower than that in the Muskingum Valley to the south.

The following table shows the approximate gradient of the main streams and their more important tributaries:

Approximate gradient of principal streams of Muskingum drainage basin.

Stream.	From—	To—	Distance in miles.	Total fall in feet.	Fall per mile in feet.
Muskingum	Mouth	Zanesville	70	106	1.5
Do	do	Coshocton	99	180	1.8
Do	Zanesville	do	29	74	2.6
Muskingum and Tuscarawas.	Mouth	Headwaters	211	530	2.5
Tuscarawas	Coshocton	Canal Dover	47	110	2.4
Do	do	Headwaters	112	350	3.1
Stillwater	Mouth	Tippecanoe	16	23	1.4
Do	do	Headwaters	47	392	8.6
Licking	Zanesville	Newark	23	134	5.8
Do	Newark	Headwaters	33	399	12
Do	Zanesville	do	56	533	9.5
Wills	Mouth	Cambridge	48	69	1.5
Do	Cambridge	Headwaters	35	201	5.7
Do	Mouth	do	81	370	4.5
Walhonding	Coshocton	Headwaters of Killbuck	54	230	4.3
Do	do	Headwaters of Black Fork.	92	635	6.9
Do	do	Headwaters of Kokosing.	62	365	5.9

From the above table it is seen that the fall per mile along the main drainage line, the Muskingum and Tuscarawas, is very small and the flow sluggish. This is especially true of Muskingum River and the lower part of the Tuscarawas. The fall per mile of the lower part of both Stillwater and Wills creeks is in keeping with their lack of current, but the fall is so great in the hills which form their headwaters that the average for the whole stream is brought up to a rather high figure.

This same feature is shown, but in a less marked degree, in all the streams, and no doubt it is gradually increasing as the water cuts back into the hills at the headwaters and spreads débris out over the valleys below. This general silting up of the lower channels is greatly prevented by the great number of dams in the streams, each one of which holds back from every flood a large amount of sediment that would have been distributed along the whole channel. Tiling also has decreased the amount of sediment reaching the streams. In the hilly portions of the basin many good farms have been ruined, and in some of the abandoned mining districts, where the timber has been cut away and the ground exposed, the hills have been cut into by

such enormous gullies that it seems as if it would be impossible ever to reclaim them. Nature will require many years to restore these waste lands unless she is intelligently assisted by man.

In the northern and western parts of the drainage basin the fields are well tiled and ditched. In fact, nearly all the swamp land has been

FIG. 5.—Map of drainage basin of Muskingum River.

reclaimed, the only extensive area still in existence being a strip along Killbuck Creek between Wooster and Shreve. All through the drainage basin there are areas that are swampy in wet years, but as a rule the drainage of the basin is good—too good, in fact, for the preservation of the summer flow of the streams.

FLOW OF STREAMS.

In connection with the United States Geological Survey, the Ohio State board of health established gages for reading the daily height of the streams at Mexico and Fremont, on the Sandusky, and at Waterville, on the Maumee; but owing to changes in the stream bed and certain obstructions to the flow, it was not possible to work up the discharge of the Sandusky at Fremont, and the flow of this stream at Sandusky and of the Maumee at Waterville only are given.

SANDUSKY RIVER.

The daily gage-height readings of Sandusky River at Mexico extend from November, 1898, to November, 1900, when the gage was destroyed

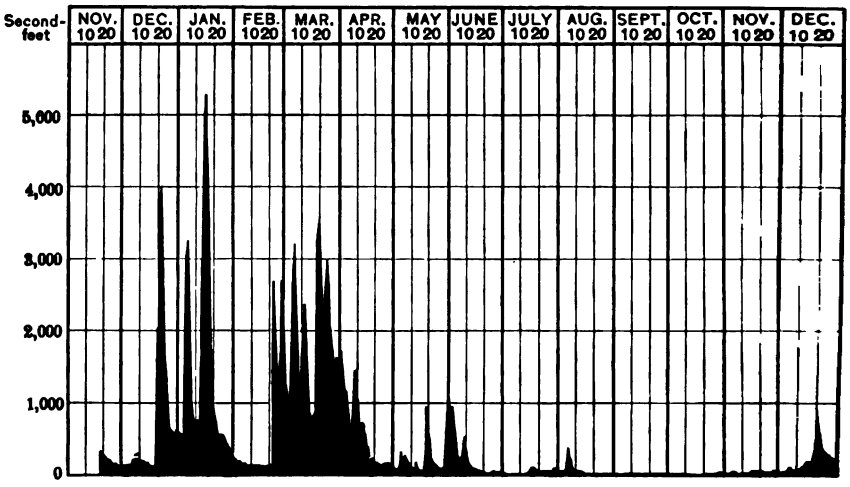

FIG. 6.—Diagram showing discharge of Sandusky River at Mexico in 1898–99.

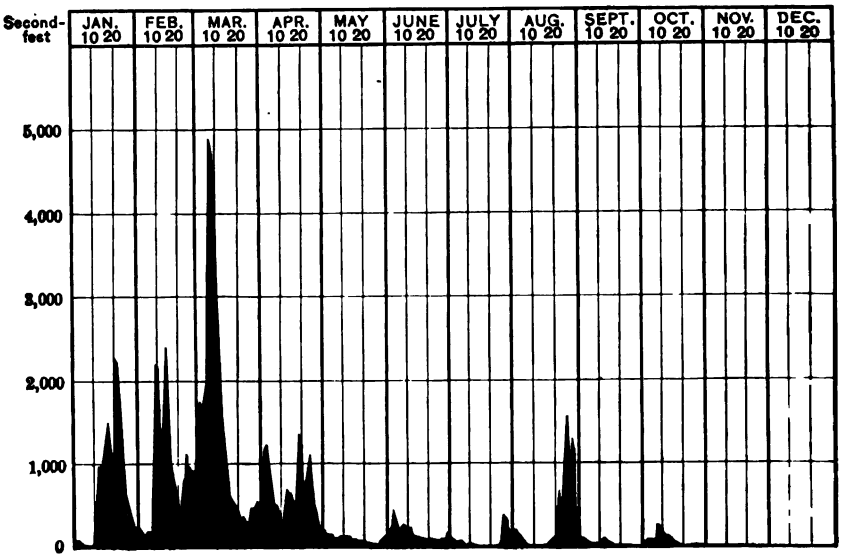

FIG. 7.—Diagram showing discharge of Sandusky River at Mexico in 1900.

and the station abandoned. Owing to lack of facilities, it was impossible to estimate the rainfall by any means except that of averaging the monthly records of the various stations in and near the drainage basin. For the basin above Mexico six stations are used, and in the same table there are given two other stations, with a separate average of the eight for use at such time as it is possible to get the discharge at the Fremont station.

Monthly discharge of Sandusky River at Mexico, 1898–1900.

[Drainage area, 776 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.			
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.	Rainfall in inches.	Ratio.
1898.								
December	4,085	100	727.4	44,726	0.950	1.090	2.65	0.411
1899.								
January	5,410	240	1,515.6	93,222	1.953	2.253	3.29	.684
February	2,725	100	636.4	35,344	.820	.854	2.21	.386
March	3,580	795	1,871.1	115,049	2.424	3.714	4.84	.767
April	1,730	100	600.2	35,714	.773	.856	1.44	.594
May	1,145	45	626.6	13,963	.292	.384	4.55	.073
June	995	6	207.5	12,347	.267	.297	2.95	.100
July	100	4	31.7	1,949	.040	.040	4.82	.008
August	375	3	43.9	2,699	.056	.066	1.86	.035
September	6	2	2.9	173	.003	.003	2.34	.001
October	30	2	8.9	547	.011	.011	2.22	.005
November	45	12	23.9	1,422	.030	.030	2.28	.013
December	655	30	170.8	10,502	.220	.230	3.20	.072
Year	5,410	2	496.6	322,901	.640	8.688	36	.241
1900.								
January	2,245	8	655.5	40,581	.844	.974	2.22	.439
February	2,380	125	888.9	49,367	1.145	1.192	4.18	.285
March	4,870	270	1,407	86,513	1.813	2.098	2.64	.798
April	1,355	180	686.2	40,882	.884	.984	2.84	.340
May	180	20	88.4	5,436	.114	.134	2.57	.521
June	445	80	161.5	9,610	.208	.228	3.94	.578
July	410	6	78.7	4,839	.101	.121	4.72	.256
August	1,560	4	242.6	14,917	.312	.362	4.76	.076
September	125	4	38.3	2,279	.049	.058	2.04	.028
October	270	8	70.1	4,310	.090	.100	2.53	.039

The mean monthly rainfall shown in the next to last column was obtained from the following table, the data for which were obtained from the reports of the United States Weather Bureau:

Rainfall in Sandusky drainage basin, 1898–1900.

	1898.	1899.												1900.										
	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Total.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Total.
Bucyrus	1.25	.....	.....	.....	1.33	5.71	2.95	5.01	0.40	3.88	.....	.....	.....	.....	0.60	2.85	2	.....	3.27	4.64	3.40	4.63	2.10	1.75	1.08	0.73	.....
Findlay	2.51	3.19	1.50	4.57	1.77	3.31	2.30	5.08	.79	1.61	1.91	1.71	3.06	30.80	1.70	5.19	1.52	1.99	2.86	1.60	5.84	6.27	1.50	3.16	3.10	.08	35.61
Kenton	3.04	3.46	2.11	4.90	1.21	4.65	1.05	3.28	2.25	1.32	2.05	3.04	2.93	32.25	3.11	4.29	2.31	2.30	1.39	5.04	3.67	3.41	1.52	3.67	4.66	1.01	36.38
Mansfield	4.30	3.61	2.31	5.55	1.75	4.60	6.42	4.75	1.60	4.55	3.20	2.40	3.39	44.13	2.89	4.75	4.30	4.85	3.50	4.15	7.88	4.65	2.23	2.15	4.63	1.62	47.60
Marion	1.65	2.97	1.68	4.71	1.45	3.05	2.42	4.73	3.47	1.39	1.79	2.29	3.02	32.97	3.35	3.86	2.74	3.12	1.33	5.64	3.13	4.52	2.99	1.84	3.45	1.31	37.01
Upper Sandusky	3.14	3.20	2.44	4.48	1.14	5.98	2.55	6.08	2.67	1.30	2.16	1.95	3.60	37.53	2.69	4.14	3.25	2.20	3.05	2.57	4.37	5.06	1.98	2.62	2.57	1.35	35.85
Mean for Mexico station	2.65	3.29	2.21	4.84	1.44	4.55	2.95	4.82	1.86	2.34	2.22	2.28	3.20	36.00	2.22	4.18	2.64	2.89	2.57	3.94	4.72	4.76	2.04	2.53	3.24	1.15	36.88
Tiffin	3.30	3.72	2.20	5.06	2.64	5.42	2.10	4.56	.90	1.53	2.15	1.84	3.27	35.39	2.19	4.91	2.14	2.55	1.21	3.80	7.55	4.39	.91	1.91	2.90	.92	35.38
Vickery	2.43	3.23	1.49	4.87	.82	5.27	1.96	3.25	1.38	.85	1.98	1.71	2.18	28.99	1.74	3.94	1.68	1.72	2.36	3.96	5.89	2.72	1.03	1.46	2.58	.57	29.05
Mean for Fremont station	2.70	3.34	2.10	4.88	1.51	4.75	2.72	4.59	1.68	2.05	2.18	2.13	3.06	34.99	2.16	4.24	2.46	2.68	2.36	3.85	5.22	4.46	1.77	2.32	3.12	1.05	35.69

Above Mexico Sandusky River has a drainage area of 776 square miles. The country consists of rolling farm land in a high state of cultivation. There remain very few tracts of timber of large size, and the fields are fairly well drained by ditches and tile.

There are six small dams in the river above the gaging station, but they store very little water and seem to have no perceptible influence on the flow at Mexico.

MAUMEE RIVER.

The gaging station on this stream was placed at Waterville, the first available place above the influence of backwater from Maumee Bay, in November, 1898, and was abandoned early in 1902. The mean monthly rainfall in this basin was obtained by averaging the depth reported from 21 stations within or neighboring to the basin.

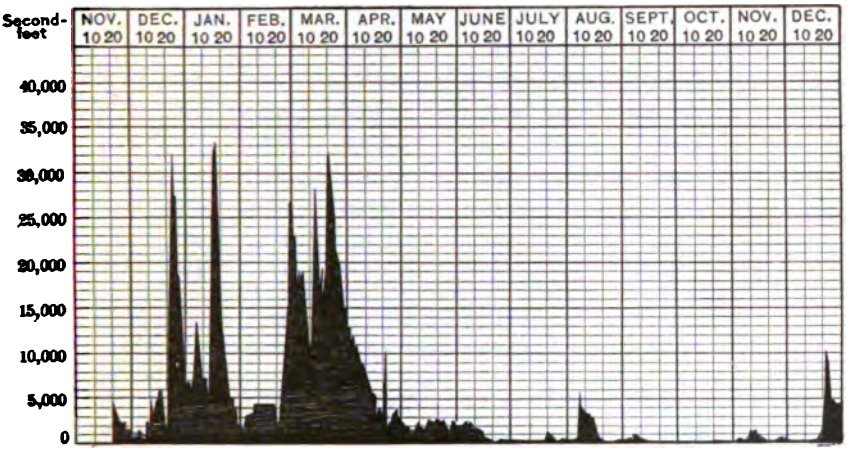

FIG. 8.—Diagram showing discharge of Maumee River at Waterville in 1898-99.

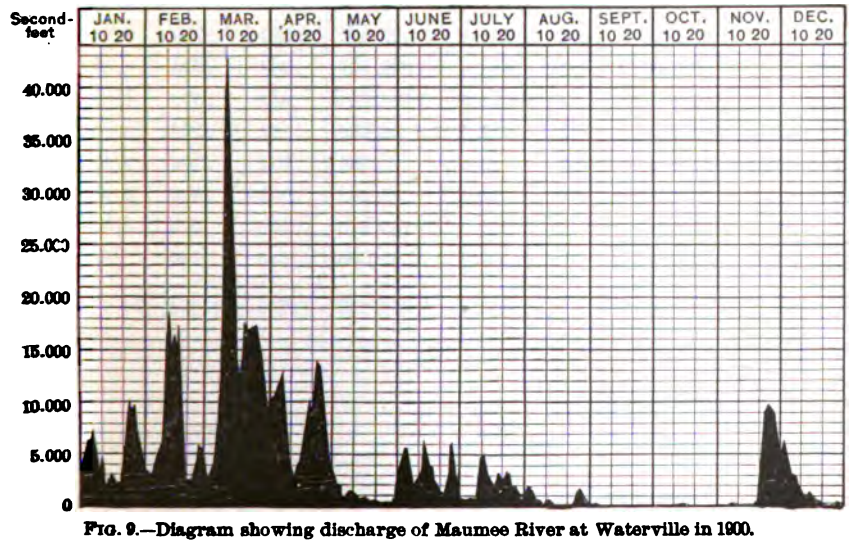

FIG. 9.—Diagram showing discharge of Maumee River at Waterville in 1900.

Monthly discharge of Maumee River at Waterville, 1898–1900.

[Drainage area, 611 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.			
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.	Rainfall in inches.	Ratio.
1898.								
December	32,120	680	8,203.4	504,407	1.325	1.525	2.41	0.632
1899.								
January	33,250	780	10,979	675,072	1.796	2.070	2.60	.796
February	26,765	1,310	6,301.2	349,950	1.080	1.070	2.14	.500
March	32,100	8,700	19,431.4	1,194,790	3.180	3.660	4.39	.833
April	12,960	1,800	6,027.5	358,661	.986	1.095	1.11	.986
May	2,750	1,200	1,990	352,360	.325	.375	3.66	.102
June	2,450	150	1,057.3	62,814	.173	.203	1.88	.108
July	1,200	3	285.3	17,536	.046	.046	4.36	.011
August	4,940	70	1,362.2	43,758	.223	.253	2.09	.121
September	990	20	259	15,411	.042	.042	2.40	.018
October	150	5	48.1	2,955	.007	.007	2.51	.003
November	1,310	40	521	31,001	.085	.085	2.12	.040
December	9,950	10	2,323.2	142,847	.380	.450	3.08	.146
Year	33,250	3	5,211.7	3,017,155	.689	9.356	32.34	.227
1900.								
January	10,200	1,200	5,053.9	310,752	.827	.947	1.25	.757
February	18,985	2,450	7,943.9	441,180	1.300	1.342	4.50	.298
March	42,750	2,600	15,609.7	959,803	2.554	2.942	2.13	1.381
April	13,965	2,600	8,149.1	484,904	1.330	1.480	2.40	.617
May	3,050	200	937.7	57,657	1.530	1.760	2.65	.664
June	6,320	680	3,443.6	204,907	.563	.623	4.30	.145
July	5,120	300	2,093.5	128,724	.342	.392	4.89	.080
August	1,930	5	562.7	34,598	.092	.104	3.56	.029
September	250	5	26.8	1,596	.004	.004	1.45	.003
October	250	5	40.6	2,499	.006	.006	2.80	.002
November	9,700	10	2,619	155,840	.428	.478	3.89	.123
December	6,320	70	1,508.7	92,766	.246	.286	.83	.344
Year	42,750	5	4,665.7	2,275,226	7.635	10.364	34.65	.299

Rainfall in Maumee

	1898.	1899.										
	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov
Angola	2.17	3	3.17	4.17	0.65	4.43	1.02	4.04	2.77	3.21	3.32	2.27
Auburn	1.77	2.08	2.01	3.02	.69	3.62	2.02	4.73	3.73	2.17	3.27	1.80
Bluffton	2.12	2.60	1.99	4.96	.97	3.31	2.02	4.61	2.12	1.59	2.60	3.28
Fort Wayne	2.33	2.34	2.35	5.01	.70	2.83	2.29	4.28	6.06	3.49	2.30	2.75
Adrian	1.85	1.91	2.06	3.87	.59	5.57	3.35	6.08	2.10	3.24	2.74	1.85
Hillsdale	1.89	2.44	2.02	4.01	.87	6.65	3.02	3.81	1.26	3.13	3.24	2.01
Benton Ridge	3.51	3.70	1.31	5.18	1.48	2.44	1.97	4.21	.66	.96	1.74	1.91
Bowling Green	1.57	2.17	2.39	4.59	1.21	3.24	2.33	4.33	.75	2.01	2.47	1.84
Celina	1.85	2.76	1.88	3.67	.70	2.92	1.67	4.06	.....	.....	2.61	2.03
Defiance	2.40	2.08	2.21	3.99	2.49	3.05	1.64	4.22	1.77	3.71	2.63	2.06
Findlay	2.51	3.19	1.50	4.57	1.77	3.31	2.30	5.08	.79	1.61	1.91	1.71
Hedges	2.50	2.88	1.54	.....	.....	2.37	.....	.....	.....	.....	.....	.....
Kenton	3.04	3.46	2.11	4.90	1.21	4.65	1.05	3.28	2.25	1.32	2.05	3.04
Leipsic	2.58	2.57	1.82	4.62	.56	.....	1	.....	2.50	1.50	2.80	1.63
Montpelier	2.55	2.09	2.01	3.41	1.92	3.65	2.05	.....	.....	.....	.....	.....
Napoleon	3	1.25	2.52	4.39	1.56	2.68	1.51	5.19	.85	3.60	2.24	1.63
New Bremen	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Ottawa	2.48	2.84	1.58	4.52	1.45	2.94	1.65	3.25	2.85	1.86	2.36	1.88
Swanton	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Van Wert	2.25	2.25	1.70	4.76	.44	.....	1.80	6	.....	2.35	.....	.....
Wauseon	3.28	3.90	4.52	5.47	1.19	4.53	1.11	4.52	.84	2.70	2.50	2.17
Total	45.71	49.46	40.69	79.11	19.95	62.21	33.80	71.69	31.30	38.45	40.19	33.86
Mean	2.41	2.60	2.14	4.39	1.11	3.66	1.88	4.36	2.09	2.40	2.51	2.12

basin, 1898–1900, in inches.

1899.		1900.												
Dec.	Total.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Total.
3.31	35.65	1.37	5.23	2.88	2.21	2.23	4.60	8.02	3.68	1.50	4.35	5.58	0.65	42.48
1.92	31.01	.76	5.05	1.89	1.98	2.44	5.61	5.29	3.87	1.39	2.44	4.45	.45	35.62
2.92	32.47	.83	3.95	1.92	2.92	3.34	5.01	3.03	4.15	1.87	2.74	3.88	.93	34.57
3.16	37.56	.86	5.11	1.87	2.26	3.40	6.05	3.52	.....	1.84	2.83	.....	.....	.....
2.39	35.75	.72	3.26	2.91	1.85	3.33	3.15	7.04	2.50	2.12	2.83	3.90	.27	33.88
2.77	35.23	.79	4.09	2.55	2.22	2.35	4.49	8.42	1.80	1.55	2.99	4.80	.43	36.48
3.39	28.95	1.67	5.50	2.81	2.72	2.42	2.54	4.45	4.69	1.66	3.05	3.66	1.18	36.35
2.70	30.03	.97	3.36	1.33	2.45	2.95	3.80	3.76	4.24	.85	2.95	3.32	1	30.98
4.01	.....	1.58	4.99	2.21	2.05	2.17	4.07	2.33	4.18	1.51	1.87	2.43	1.33	30.72
3.50	33.35	1.20	4.41	1.81	3.17	3.21	5.36	6.03	2.58	1.07	3.07	3.76	.59	36.26
3.06	30.80	1.70	5.19	1.52	1.99	2.86	1.60	5.84	6.27	1.50	3.16	2.10	.88	35.61
.....	36.26	.....	3.11	1.14	2.35	2.89	4.22	4.75	2.85	.92	2.78	4.10	.72	.....
2.93	32.25	3.11	4.29	2.31	2.30	1.39	5.04	3.67	3.41	1.52	3.67	4.66	1.01	36.38
.....	.....	.....	.....	.....	.....	.....	.....	4.80	2.45	.70	1.55	.....	.75	.....
.....	.....	.96	4.44	2.39	2.23	2.09	6.61	6.35	2.16	1.63	3.62	3.92	.57	36.97
3.48	30.90	.65	4.33	2.79	2.50	2.30	5.47	3.48	2.80	1.34	2.62	3.61	.95	32.84
.....	.....	1.12	.....	.....	1.74	2.28	3.91	2.37	4.09	1.84	2.19	4.22	1.09	.....
3.26	30.44	1.42	4.55	1.67	2.21	2	2.89	3.88	3.10	1.11	2.03	3.37	1.05	29.34
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	3.32	1.03	.....
.....	.....	.....	.....	.....	.....	.....	.....	.....	5.74	1.55	2.12	3.37	.....	.....
3.30	36.81	1.50	5.57	2.17	4.04	4.12	3.01	5.71	2.97	1.61	3.09	4.48	.93	39.38
46.16	.....	21.21	76.43	36.17	43.19	47.77	77.43	92.74	67.71	29.08	55.95	73.93	15.81	.....
3.08	32.34	1.25	4.50	2.13	2.40	2.65	4.30	4.89	3.56	1.45	2.80	3.89	.83	34.65

The rainfall table for 1901 gives the mean annual rainfall for the basin as 30.89 inches, which is 1.45 inches less than the rainfall for 1899 and 3.76 less than for 1900. This lower rainfall is accompanied by a falling off in the run-off, the figure being 19.9 per cent, as against 22.7 per cent and 29.9 per cent for 1899 and 1900, respectively.

To prevent overvaluation of these results, it must be noted that the canal system takes a small amount of water from above the gaging station and returns some of it below this station. On the extremely low flows this may have some influence, but it is not believed that it is extensive.

Monthly discharge of Maumee River at Waterville in 1901.

[Drainage area, 6,111 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.			
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.	Rainfall in inches.	Per cent.
January	6,110	480	2,389.7	146,937	0.391	0.451	1.90	23.7
February	9,450	580	3,716.8	206,421	.608	.633	1.45	43.6
March	27,600	2,240	11,207.9	689,147	1.834	2.114	2.58	81.9
April	8,160	1,020	3,646.3	216,970	.596	.658	2.18	30.2
May	22,750	530	3,147.9	193,557	.515	.594	3.90	15.3
June	15,220	660	3,599	214,155	.589	.657	3.84	17.1
July	14,215	80	2,503.1	153,910	.410	.473	3.06	15.5
August	140	5	47.7	2,935	.008	.009	2.74	.3
September	80	10	42	2,499	.007	.008	1.35	.6
October	1,020	10	246.1	15,132	.040	.046	2.73	1.7
November	180	60	98	5,831	.016	.018	1.53	1.2
December	11,200	110	2,585.8	158,995	.423	.488	3.63	13.5
Year	27,600	5	2,771.2	2,006,489	.453	6.149	30.89	19.9

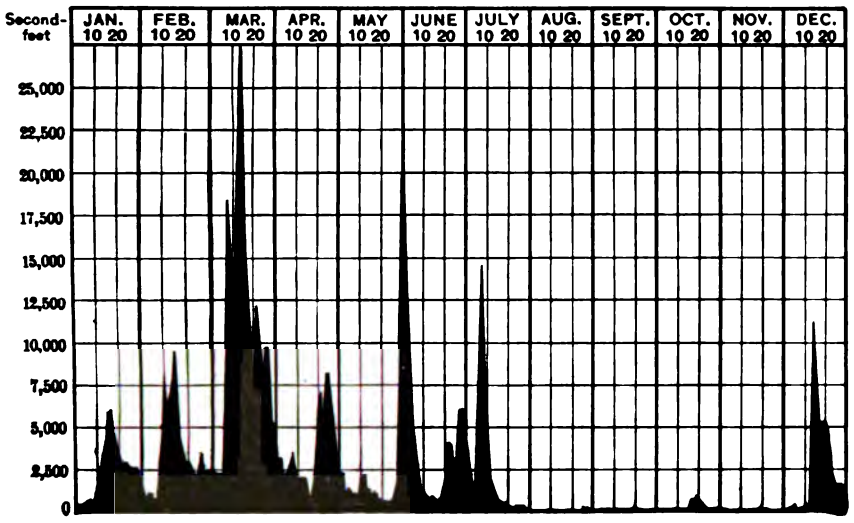

FIG. 10.—Diagram showing discharge of Maumee River at Waterville in 1901.

Rainfall in Maumee basin in 1901, in inches.

Station.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Total.
Angola	3.19	1.93	3.26	2.54	2.75	4.10	5.95	4.58	0.87	6.15	1.81	3.47	40.60
Auburn	1.58	1.44	1.15	1.94	2.45	2.87	1.38	3.71	1.30	3.71	1.15	1.57	24.25
Bluffton	1.17	.93	2.76	2.34	4.16	3.29	1.23	4.79	1.12	3.40	1.50	4.01	30.70
Fort Wayne	1.49	1.74	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Adrian	2.12	1.31	2.37	1.33	2.53	2.23	1.75	2.05	.97	3.46	1.89	2.91	24.92
Coldwater	2.79	1.75	2.71	2.18	1.78	4.96	4.67	4.10	.73	6.56	1.82	3.25	37.30
Hillsdale	2.53	2.63	2.95	1.98	2.64	3.55	2.68	2.20	.93	5.63	2	3.12	32.84
Benton Ridge	1.42	2.10	3.15	2.06	3.70	3.59	2.66	2.02	1.82	.24	.....	3.59	.....
Bowling Green	2.57	1.30	2.86	2.23	4.91	3.98	4.42	4.38	1.32	1.15	2.33	4.21	35.66
Celina	1.74	.80	3.14	2.27	4.79	3.96	3.24	1.86	1.95	.94	.95	3.58	29.24
Defiance	1.10	1.07	2.35	1.87	3.93	3.47	6.08	2.16	1.54	2.63	1.41	4.14	31.75
Findlay	1.46	1.45	3.30	2.46	4.65	2.79	1.86	2.96	1.85	.86	.94	3.87	37.95
Hedges	1.12	1.08	3.02	2.19	5.28	3.26	1.89	2.25	1.76	2.27	1.35	4.03	29.45
Kenton	2.61	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Leipsic	1.33	1.70	2.20	1.81	3.95	3.95	3.20	2.14	.....	1.22	1.01	4.43	.....
Montpelier	3.30	1.70	2.44	2.32	3.52	4.01	5.07	2.45	.82	4.09	1.71	3.10	35.03
Napoleon	1.58	1.25	2.51	2.01	4.03	5.10	2.98	2.53	.64	1.73	1.27	3.02	28.65
New Bremen	1.28	.77	1.26	2.11	4.67	5.14	1.68	2.11	1.44	1.09	1.25	3.75	26.55
Ottawa	1.64	1.26	2.60	2.34	4.32	2.46	3.32	2.61	1.61	.59	1.64	2.25	27.64
Swanton	2.01	1.44	2.05	2.12	3.58	6.68	3.71	2.55	1.32	1.36	1.92	3.88	32.62
Vanwert	1.68	1.46	2.04	2.11	6.63	2.32	1.63	1.50	1.74	2.24	1.28	4.64	29.27
Wauseon	2.10	1.38	3.57	2.99	3.67	5.15	2.33	1.93	1.95	2.88	1.61	5.73	35.32
Total	41.81	30.44	51.69	43.67	77.94	76.88	61.26	54.28	25.68	51.96	29.14	72.55	.....
Mean	1.90	1.45	2.58	2.18	3.90	3.84	3.06	2.74	1.35	2.73	1.53	3.63	30.89

The Maumee basin has an area of 6,111 square miles, portions of which are in Indiana and Michigan. The central and southern parts of the district are rather level, while the northern part is rolling. In portions of the basin, notably the central part, there are still extensive areas of scrub-timber land; but by far the largest part is under cultivation or cleared for grazing.

MUSKINGUM RIVER.

No gaging station was established on Muskingum River, but a method by which the discharge could be approximately determined was found and used for one year. In fact, an accurate gaging of this stream with a reasonable expenditure of time and money would be very difficult. Below Zanesville slack-water navigation is maintained by ten fixed dams. Boats are locked through these from pool to pool or from level to level, as in a canal system. This effectually prevents the use of a current meter on the main portion of the river and makes it necessary to go to the tributaries for such work. To do this would involve the maintenance of a large number of gages and the making of innumerable meter measurements. This plan not being practicable, it was determined to approximate the flow by computations of

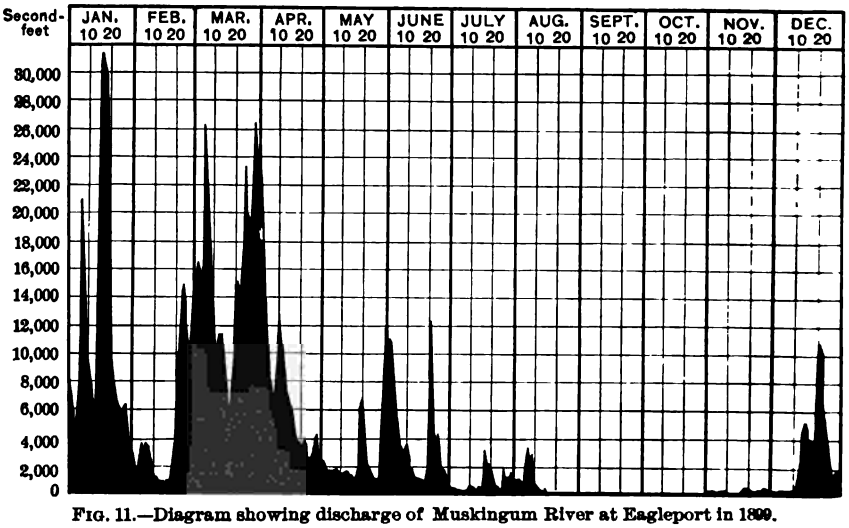

FIG. 11.—Diagram showing discharge of Muskingum River at Eagleport in 1899.

the discharge over one of the dams. Owing to the irregularity in the crest of these dams, to leakage, waste through locks, and other causes, it was found impossible with the time at hand to do as good work as the method would warrant.

Dam No. 8, at Eagleport, was selected as affording the best opportunity for success. The pool levels both above and below this dam, together with plans and other valuable information, were obtained through the courtesy of Mr. Edmund Moeser, United States resident engineer, Zanesville, Ohio. From the pool levels, which were read from one to six times daily, the daily average height of the water above the dam was computed. A formula for the flow over this dam was obtained from the Rafter experiments at Cornell University. Owing to the high water, it was impossible to survey a low place in the crest, and the effect of this lowered portion was estimated from the water powers both above and below this dam.

Estimated monthly discharge of the Muskingum River at Eagleport, Ohio.

[Drainage area, 6,849 square miles.]

Month.	Discharge in second-feet.			Run-off.			
	Maximum.	Minimum.	Mean.	Total in acre-feet.	Depth in inches.	Second-feet per square mile.	Mean rainfall in inches.
1899.							
January	31,590	2,670	11,466	705,008	1.98	1.67	2.61
February	14,970	720	5,137	341,368	.78	.75	1.88
March	26,530	5,640	15,898	978,546	2.68	2.32	4.52
April	22,570	2,530	7,146	425,213	1.16	1.04	1.81
May	12,020	1,060	2,789	171,501	.47	.41	4.76
June	12,370	720	3,945	234,742	.64	.58	4.00
July	3,520	251	960	59,007	.16	.14	4.35
August	3,500	171	709	43,612	.12	.10	1.47
September	194	171	175	10,483	.03	.03	3.30
October	348	171	181	11,141	.03	.03	2.12
November	476	194	322	19,183	.05	.05	1.60
December	11,120	220	2,711	166,724	.46	.40	3.05
The year	31,590	171	4,286	3,166,528	8.51	.627	35.42

The mean monthly rainfall as shown in the last column was obtained from the following table, the data for which were obtained from the reports of the United States Weather Bureau:

Rainfall on Muskingum basin in 1899, in inches.

Station.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Total.
Ashland	2.16	1.24	2.71	0.69	4.50	3.67	3.92	1.13	3.13	2.63	1.71	3.10	30.59
Bangorville	3.68	1.64	4.38	1.01	4.38	6.33	5.45	2.43	3.24	2.59	1.98	2.98	40.09
Canton	2.12	1.72	4.86	4.45	5.52	2.85	4.69	1.14	3.26	1.83	2.14	3.56	38.15
Green Hill	1.88	1.72	3.58	2.41	6.04	3.20	5.19	1.38	3.43	1.88	1.51	3.36	35.58
Wooster	3.29	1.04	3.95	1.28	4.42	1.95	3.73	.53	5.56	2.21	1.59	2.78	32.93
Cambridge	2.17	2.05	4.59	1.36	4.55	4.76	4.66	1.05	2.60	2.21	1.44	3.17	34.61
Canal Dover	3.28	2.18	4.53	2.17	6.84	2.89	3.53	1.28	3.61	1.65	1.90	3.59	37.45
Carrollton	2.13	2.54	4.17	2.69	5.92	3.92	4.15	1.06	.....	.....	.....	.....	.....
Granville	2.41	1.84	5.60	2.44	3.15	4.72	5.47	1.95	2.62	.....	1.90	3.19	.....
Gratiot	2.53	2.21	5.48	1.67	3.53	4.12	3.64	1.16	2.59	1.74	1.41	3.08	33.16
Milfordton	2.93	1.98	4.52	1.25	3.74	2.55	3.87	1.45	2.54	2.44	1.37	2.78	31.42
Killbuck	3.15	.89	4.51	1.35	4.74	2.82	3.50	.82	3.23	1.81	1.25	2.38	30.15
Pataskala	2.39	1.97	5.38	2.15	3.40	7.68	4.83	1.16	2.43	2.44	1.68	3.10	38.61
Philo	2.03	2.16	4.55	.96	4.45	5.08	4.71	1.39	4.37	2.40	1.10	2.86	36.06
Warsaw	3.06	1.70	5.01	1.22	6.17	3.44	4.27	4.08	3.58	1.76	1.26	2.70	38.25
Total	39.21	27.48	67.82	27.10	71.35	59.98	65.31	22.01	46.19	27.59	22.33	42.63	.....
Mean	2.61	1.83	4.52	1.81	4.76	4.00	4.35	1.47	3.30	2.12	1.60	3.05	35.42

The fifteen stations given here are scattered over the whole drainage area above Eagleport, and the mean of the rainfall at these stations will give a fair average for the basin.

Even if the computations for the discharge of this stream were accurate, it would be impossible to make a proper study of the run-off of this drainage basin, as the canal system diverts a portion of the flow of the northern tributaries to the Cuyahoga River and of the southwestern tributaries to the Scioto River. From the data at hand it is impossible to state the extent of this diversion and the effect it should have on the run-off, but it should be more noticeable in the low flows than in the higher ones, as a larger per cent of the former is used.

From the above causes the figures in the preceding table showing the run-off are of secondary value. It is shown that 8.51 inches of rainfall, or 21.2 per cent, constituted the run-off. In the survey of the canals of the State by the United States Engineering Department, in 1895, estimates were made of the run-off of this river at Zanesville. The average per cent of run-off for eight years (1888–1895) was 33, or 11.8 per cent above the estimate for 1899. The average for three dry years, 1889, 1894, and 1895, was 22 per cent, or 0.8 per cent above that for 1899. These comparisons show that the figure for 1899 is probably too low.

From the middle of August until the last of October the water never rose above the true crest of the dam, and the constant flow shown in the chart is due to the flow over the depressed crest, leakage, locking of boats, etc. The methods used in the work would not admit of great refinement in the estimate of these low flows, so an average figure was obtained for them from the water-power ratings, as mentioned above.

DAMS AND WATER POWER.

The traveler through the older portions of Ohio has his attention arrested by the desolateness of the old water-power mills. On every hand there are to be found the ruins of some once busy mill, its dam broken and its race clogged and overgrown with weeds. At no very distant time these small mills entered very largely into the daily life of a good portion of the people, not from the importance of the mills themselves, but from the fact that they supplied the daily necessities of life, especially in the more isolated districts. The usual grist and saw mill was here supplemented by small woolen mills, cooper shops, an occasional machine or wagon shop, and rather numerous cider presses and cane mills.

As would be expected, this method of obtaining power was of more importance in the hilly eastern section of the State than in the more level district of the north and west. At one time in the Muskingum basin water powers were to be found on many small streams, the back-

water of one dam reaching to the tailrace of the power above. These numerous dams made the flow of the stream more uniform than ever, and frequently gave steady power throughout the year. Evidence of the general use of water power is not to be found in any of the other three drainage areas, though the flow of Sandusky River proper was formerly well utilized. In the Muskingum, Maumee, and Miami drainage basins the canal system furnished considerable water power.

Within the last few decades there has been a decline in the amount of water power, for several reasons. The most important of all, of course, has been the increasing lack of uniformity in the run-off of the streams, due to the rapid deforesting of the tributary areas and the ever-increasing amount of tiling and ditching of agricultural land. From this cause alone many mills that formerly gave full power for twelve months in the year can be used now for but six or eight months.

In this state of affairs it has become unprofitable to tie up capital in a plant from which no certain return can be expected, and many owners were led either to abandon their water powers altogether and put in steam plants or, if the power were still good for short periods, to put in an auxiliary steam plant to tide over seasons of low water. The gradual reduction in price of coal made this scheme more and more economical. Another cause of the disuse of water power is probably to be found in the decline of water transportation, which deprived many of the mills of their raw material and of a market for their produce. This applies especially to the canal powers of the Muskingum and Maumee basins. Just as a decline in water transportation was inimical to these mills, so was the increase in rail transportation. The cheapening of this latter method of transportation made it possible for large concerns to underbid in the local markets the products of the local mills.

The many unfavorable features to be contended with in the operation of a water-power mill have tended to prevent the expenditure of money for the repairs needed, and these properties have gradually run down to such an extent that they can no longer be operated.

The silting up of the pond areas has also worked against the successful operation of many of the mills.

On the other hand, there have appeared in the last few years conditions which will in time increase the amount of water power developed, probably not at numerous small plants, but at large central stations. The transmission of power by electricity, in connection with a general rise in the price of coal, will tend more and more to bring about the larger use of water power. Even now there are four plants in these drainage areas installed for the development of electrical power, one each in the Sandusky and Muskingum and two in the Maumee drainage basin.

SANDUSKY BASIN.

All the water power in this drainage area is to be found on Sandusky River proper. Its tributaries are too small to furnish profitable power.

Following is a list of the dams on this stream, with notes as to their use. No information is at hand as to the amount of power furnished:

Table showing dams on Sandusky River.

No.	Name.	Distance from mouth of river.	Approximate location.	Use.	Width in feet.	Height in feet.	Length of backwater in miles.	Pond area in acres.
1	McMickles	96	1½ miles above Bucyrus	Watering stock	60	2.5	3	20
2	Waterworks	95	Just above Bucyrus	Water supply	50	1.5	1	8
3	do	94½	In Bucyrus	do	70	2.5	.5	4
4	Gas works	94	do	Gas works	70	.4	.25	3
5	Waterworks	70	In Upper Sandusky	Water supply	70	2.5	.1	.6
6	Indian Mill	68	Below Upper Sandusky	Grist mill	60	.7	1	6
7	St. Johns	45	9 miles above Tiffin	do	175	.3	1	20
8	Waterworks	38	Just above Tiffin	Water supply	175	5.5	4	30
9	Pioneer Mills	37	In Tiffin	Grist mill	250	7	1	45
10	Spec	36½	do	do	225	8	.3	14
11	Beckley	36	Just below Tiffin	do	250	4	.5	15
12	Bakers	34½	2 miles below Tiffin		250	4	(a)	
13	Umsted	32½	3½ miles below Tiffin	Grist mill	200	6	1	25
14	Fort Seneca	28	8 miles below Tiffin	Tile mill	200	5	1	25
15	Ballville	18	2½ miles above Fremont	Grist mill	300	14	1.1	30
16	First dam	17	1½ miles above Fremont	Electric power	300	14	.5	15

a Broken out.

The St. Johns, Pioneer Mills, Spec, and Ballville dams furnish power for feed and flour mills. Enough power is furnished to run nearly the year round, especially for the lower mills.

The Tiffin waterworks dam supplies power enough to run a 1,500,000-gallon pump, working against about 115 feet head, for eight or ten hours each day.

The "First dam," above Fremont, furnishes an extensive and reliable power for practically the whole year.

MAUMEE BASIN.

The dams in this basin are confined to two on Maumee River, for the supply of the Miami and Erie Canal; one on Blanchard River, to impound water for Findlay, and a few on St. Joseph River, in Indiana, for water power. The first State dam is at Grand Rapids. It is 1,610 feet long, 5½ feet high, and backs up the water to Napoleon, 19 miles

above. The second is below Defiance. It is 400 feet long, 7 feet high, with flashboards 1 foot high, and backs the water up into Defiance, 5 miles above.

The dam at Findlay is 200 feet long, 8 feet 3 inches high, and backs up the water 3 miles. It is used to divert the water of Blanchard River to a filtering basin, from which it is pumped by steam for public supply.

The Miami and Erie Canal furnishes power to run a flour mill and sawmill at Waterville, and a flour mill, paper mill, and two electric-power houses at Maumee. The fall is obtained between the canal and Maumee River.

MIAMI BASINS.

The decline in the dams in the Little Miami drainage basin is only apparent in the smaller number of mills, as the horsepower in use has diminished but little owing to the more complete development of some of the larger powers, especially the one at Kings Mills.

The data concerning the Miami dams and water powers were obtained by personal inspection and from reports of the owners and operators. The estimates on the pond areas are crude and are given only as a guide to the amount of storage available. The areas of tributary drainage basins were obtained by planimeter measurements from the State board of health maps of Ohio.

Dams and water power

Number.	Stream.	Dam.						
		Approximate location.	Character.	Height in feet.	Length in feet.	Condition.	Length of backwater, in miles.	Approximate pond area in acres.
1	Great Miami River	New Baltimore						
2	Fourmile Creek	Oxford	Log and slabs	2½	100	Poor	¼	3
3	Sevenmile Creek	Sevenmile	Pile and frame	8	140	Fair	¼	4
4	Great Miami River	Hamilton	Brush, stone, and timber.	4	2,000	Fair	1	200
5	do	Woodsdale	Timber frame	6	600	Good	1	70
6	do	Middletown	do	11	450	Fair	2	100
7	Twin Creek	South of West Alexandria.	Log and timber	2½	150	Good	¼	2
8	do	West Alexandria	Timber frame	5	225	Fair	¼	6
9	do	Lewisburg	Log and stone in 3 section.	3	342	Good	¼	5
10	Great Miami River	Franklin	Timber	6	600	Fair	1½	75
11	do	West Carrollton	Timber frame	10	300	Good	1½	50
		Miamisburg						
		do						
		do						
12	Mad River	Dayton	Pile and frame	4	250	Poor	¼	6

of Great Miami basin.

Area of tributary watershed in square miles.	Use of dam; if water power, kind of mill.	Water power. Average head available.	Water power. Kind of wheels.	Total horsepower claimed.	Approximate time available.	Remarks.	Number.
					Mos.		
3,734	Flour mill	4		30			1
70	Feed and saw mill	10	2 turbines	20	8	Little used	2
139	Planing mill	14	One 25-inch Victor turbine.	30	8		3
3,521	Paper mill	26		200	11	Auxiliary steam.	4
	Foundry	26	1 turbine	25	11		
	Marble works	26	Overshot	25	11		
	Agricultural implements.	18	Turbine	12	11	Auxiliary steam.	
	Bending works	18	do	12	11	do	
	Planing mill	18	do	12	11		
	Turning shop	13	Overshot	50	11	Auxiliary steam.	
	Printing shop	18	Turbine	50	11		
	Planing mill	18	do	12	11		
	Machine shop	18	do	12	11	Auxiliary steam.	
	do	13	do	12	11	do	
	do	13	do	12	11	do	
3,508	Sawmill	9	50 and 37 inch turbines	75	10	Little used	5
	Flour mill	9	Two 42 and 50 inch turbines.	100	10	do	
3,060	Canal feeder and hydraulic waterworks.	17½	One 56-inch American turbine.	85	12	Auxiliary steam.	6
	Paper mill	17½	Two 36-inch Hercules turbines.	300	11	do	
	do	17	One 36-inch Leffel turbine.	112	11	do	
	Flour mill	17	One 28-inch American turbine.	50	11	do	
	Machine shop	17	One 28-inch turbine	21	11	do	
	Paper mill	18	One 32-inch Victor turbine.	90	11	do	
160	Flour mill	8	One 26½ and one 35 inch turbine.	27	8	do	7
155	do	8½	Two 36 and one 40 inch turbine.	50	8		8
90	do	9	2 turbines	35	6	Auxiliary steam.	9
2,677	Paper mill	14½	5 turbines	100	11	do	10
	Planing mill	14	2 turbines	25	11		
2,610	Canal feeder and hydraulic.						11
	Paper mill	17½	36 and 17 inch Victor turbines.	160	10	Auxiliary steam.	
	do	17	One 35-inch Victor turbine.	150	10	do	
	Planing mill	17	One 20-inch turbine	20	10		
640	Canal feeder					Supplies some water for hydraulic.	12

Dams and water power of

Number.	Stream.	Dam.					Length of backwater in miles.	Approximate pond area in acres.
		Approximate location.	Character.	Height in feet.	Length in feet.	Condition.		
13	Mad River	Dayton	Stone fill	6	150	Fair	¼	4
14	do	Harshmanville	Stone fill, 3 sections.	5	850	Fair	¼	30
15	Spring Run	Fairfield	Earth					2
16	Mad River	Osborn	Timber	5	170	Fair	¼	10
17	do	Durbin	Brush and stone	3	100	Poor	¼	2
18	do	Springfield	Stone fill	4	75	Poor	1	8
19	Buck Creek	do	Cut stone	7	150	Fair	¼	8
20	do	do	do	4	75	Fair	¼	3
21	do	New Moorefield	Stone fill	1	30	Poor	1/16	
22	Mad River	Springfield	do	6	70	Fair	¼	4
23	do	Below Tremont	do	1½	150	Fair	¼	6
24	Chapman Creek	Tremont	Brush, stone, and plank.	3	60	Poor	¼	2
25	do	West of Tremont	Plank	3	50	Fair	¼	2
26	Mad River	South of Urbana	Log and stone	3	125	Poor	¼	4
27	Kings Creek	Kings Creek	Creek diverted three-fourths mile above.					
28	Great Miami River	Dayton	Log and stone	8	630	Poor	1	70

Great Miami basin—Continued.

Area of tributary watershed in square miles.	Use of dam; if water power, kind of mill.	Water power. Average head available.	Water power. Kind of wheels.	Water power. Total horsepower claimed.	Water power. Approximate time available.	Remarks.	Number.
					Mos.		
653	Paper mill	16	Overshot	130	10	Auxiliary steam.	13
	Cereal mill	16	do	30	10	do	
	Linseed-oil mill	16	do	25	10	do	
	Flour mill	16	1 turbine	15	10		
	Plow works	16	Overshot	6	10		
	Flour mill	16	do	9	10		
	Flax mill	16	do	9	10		
	Plow works	16	do	6	10		
	Flour mill	10	One 22-inch turbine	6	10	Auxiliary steam.	
	Tag works	10		30	10		
	Scale works	10		35	10	Auxiliary steam.	
	Flour mill	10	3 overshots, 10-foot diameter.	30	10	do	
	Linseed-oil mill	10		16	10		
	Brewery	10		12	10	Auxiliary steam.	
	Foundry and machine shop.	8	One 45-inch American turbine.	25	10	Auxiliary steam (canal to river).	
685	Corn shelter	8	One 48 and one 36 inch turbine.	50		Little used	14
	Grain elevator	8	One 48-inch turbine	25		do	
	do	8	One 8-foot reaction	20		do	
	Shoe factory	7	Three 48 and 30 inch turbines.	130	12	Only 40 horsepower used.	
	Sawmill	8	Four 36-inch turbines	75		Little used	
8	Feed mill	12	One 30-inch turbine	15		do	15
575	Flour mill	8	One 72-inch Leffel turbine.	76	11	2 short stone sections to dam.	16
	do	8	One 35-inch Leffel turbine.	35	8	Auxiliary steam.	
483	Not used						17
447	Flour mill	8	Turbines	40	8	Little used	18
150	do	24	One 42-inch Leffel turbine.	100	9	Auxiliary steam.	19
	Machine shop	22	do	80	12	do	
	Not used for power						20
82	Feed and flour mill	10	One 32-inch Leffel turbine.	23	6		21
250	Feed mill	8	Two 6-foot reaction	20	10		22
	Flour mill	8	Two 24 and 32 inch turbines.	40	10		
276	do	7	One 48 and one 40 inch turbine.	50	9		23
24	do	16	Overshot; 16-foot diameter, 5½ breast.	30	6	Little used	24
18	Feed mill	14	Overshot; 14-foot diameter, 8-foot breast.	25	6	do	25
165	do	4	2 turbines	20	6	do	26
27	Feed and flour mill	14	Overshot and 20-inch turbine.	50	9		27
1,811	Machine shop	8	3 Stillwell-Bierce turbines.	90	9	Auxiliary steam.	28

Dams and water power of

Number.	Stream.	Dam.						
		Approximate location.	Character.	Height in feet.	Length in feet.	Condition	Length of backwater in miles.	Approximate pond area in acres.
29	Stillwater Creek	3¼ miles south of West Milton.	Stone fill and timber.	5	150	Poor	1	15
30	Weimer Branch	3 miles south of West Milton.	Earth	5¼		Fair		
31	Cedar Branch	1¼ miles south of West Milton.	do	10	75	Fair		4
32	Vore Branch	One-half mile south of West Milton.	Timber and stone	4	140	Fair	¼	2
33	Rutledge Branch	West Milton	Earth					1
34	Stillwater River	Above West Milton	Stone fill	3	150	Poor	¼	4
35	do	North of Ludlow Falls.	Timber	5	100	Fair	¼	5
36	do	Sugar Grove	Timber frame	5	125	Fair	1	12
37	Painter Creek	do	do	12	50	Fair	1	4
38	Stillwater River	Covington	Stone fill and concrete.	3	100	Good	1	9
39	Greenville Creek	do	Timber frame	12	125	Good	1	12
40	do	West of Covington	do	6	120	Fair	¼	6
41	do	Gettysburg	Stone fill	4	125	Poor	¼	3
42	do	New Harris	Timber frame	6	200	Good	¼	8
			Frame and stone		180	Fair		
43	do	Above Greenville	Log and stone	5	100	Fair	¼	40
44	Stillwater River	Covington	Timber	1¼	100	Fair	¼	2
45	Spring Branch	2 miles east of Troy						
46	Great Miami River	Troy	Timber frame	5	200	Good	1	15
47	do	Below Piqua	do	2¼	250	Poor		3
48	do	Above Piqua	Timber	4	250	Fair	1	25
49	do	Below Sidney	do	3	200	Fair	1	20
50	Mosquito Creek	Sidney	Stone fill	2	50	Poor	¼	1
51	do	Pasco	do	2	50	Poor	¼	1
52	Great Miami River	Port Jefferson	Timber frame	8¼	240	Good	4	40
53	do	Quincy	Stone fill	4	150	Fair	2	35

Great Miami basin—Continued.

Area of tributary watershed in square miles.	Use of dam; if water power, kind of mill.	Water power.				Remarks.	Number.
		Average head available.	Kind of wheels.	Total horsepower claimed.	Approximate time available.		
					Mos.		
618	Flour mill	8	2 turbines	40	9	Little used	29
4	do	10	1 turbine	12	8		30
6	do	56	One 10-inch Leffel turbine.	20	8		31
3	do	20	Overshot; 20-foot diameter, 3-foot breast.	12	8		32
2	Woolen mill	18	One 6-inch turbine	10	8	Little used	33
573	Flour mill	4	One 48-inch turbine	21	9	do	34
471	Plaster mill	6	One 40-inch Victor turbine.	25	10		35
456	Flour mill	6	One 30-inch and one 25-inch turbine.	35	12		36
	Sawmill	12	Two 35-inch turbines	70	8	Little used	37
453	Saw and feed mill	6	One 40-inch and one 35-inch turbine.	45	12	do	38
197	Electric-light plant	28	Two 21-inch Victor turbines.	80	12		39
192	Feed mill	6	One 48-inch Leffel turbine.	20	10		40
173	do	6	One 23-inch and one 48-inch Leffel turbine.	25	10	Little used	41
157	Flour mill	7	One Leffel turbine	40			42
110	Flour mill	5	One 40-inch turbine	20	6	Auxiliary steam	43
233	do	16	2 Victor turbines	50	8		44
	Corn sheller	16	1 turbine	25	8		
3	Sawmill	29	Turbines	25	6	Little used	45
910	Flour mill	12	One 44-inch Eclipse turbine.	75	12		46
876	Not used						47
578	Canal feeder						48
550	Flour and feed mill	11	2 Leffel turbines	50	9	Auxiliary steam	49
60	Electric-light plant	41	One 20-inch Leffel turbine.	61	10	Auxiliary gas	50
56	Flour mill	25	Four 15-inch turbines	60	8		51
452	Canal feeder						52
397	Flour mill	5½	One 45 and one 35 inch Leffel turbine.	45	12		53

Dams and water power of

CANAL WATER POWER.

Number.	Stream.	Dam.						
		Approximate location.	Character.	Height in feet.	Length in feet.	Condition.	Length of backwater in miles.	Approximate pond area in acres.
1	Miami and Erie Canal.	Hamilton						
2	....do	Excello						
3	....do	Amanda						
4	....do	Middletown						
5	....do	Franklin						
6	....do	Miamisburg						
7	....do	West Carrollton						
8	....do	North of West Carrollton.						
9	....do	....do						
10	....do	Tippecanoe						
11	....do	Below Piqua						
12	....do	....do						
13	....do	Piqua						
14	Hydraulic from canal.	....do						

Great Miami basin—Continued.

CANAL WATER POWER.

Area of tributary watershed in square miles.	Use of dam; if water power, kind of mill.	Water power.				Remarks.	Number.
		Average head available.	Kind of wheels.	Total horsepower claimed.	Approximate time available.		
					Mos.		
.....	Knitting mill	4	1 turbine	20	11	Auxiliary steam.	1
.....	Paper mill	8½	2 Hercules turbines	100	11	....do	2
.....	....do	4½	Two 54 and one 36 inch American turbines.	75	11	....do	3
.....	....do	9	2 turbines	20	10	....do	4
.....	....do	12	One 48-inch American turbine.	50	10	....do	5
.....	Flour mill	7	One 36 and one 42 inch American turbine.	40	10	....do	6
.....	Paper mill	8	One 32 and one 30 inch Victor turbine.	70	11	....do	7
.....	Flour mill	6	One 36 and one 42 inch Victor turbine.	50	11	.....	8
.....	....do	7	1 turbine	25	11	.....	9
.....	....do	9	50-inch and 21-inch turbines.	40	11	.....	10
.....	....do	9	1 turbine	25	11	.....	11
.....	Elevator	11	Overshot, 9 feet diameter, 12 feet breast.	20	11	Little used	12
.....	Flour mill	9½	One 36-inch Victor turbine.	60	10	Auxiliary gas	13
.....	Waterworks	20	25-inch and 30-inch Victor turbines.	210	12	.....	14

Dams and water power

Number.	Stream.	Dam.						
		Approximate location.	Character.	Height in feet.	Length in feet.	Condition.	Length of backwater in miles.	Approximate pond area in acres.
1	Little Miami River	Plainville	Timber	3½	300	Poor	½	25
2	East Branch, Little Miami.	Above Batavia	Log and stone fill	4½	75	Fair	½	6
3	do	Williamsburg	Natural riffle					
4	Little Miami River	Symmes	do					
5	do	Loveland	Log	4	300	Poor	1	30
6	do	Fosters	Timber	5	300	Fair	1	30
7	do	Kings Mill	do	6	900	Good	1	100
8	do	Morrow	Brush and stone	6	300	Fair	½	20
9	do	Hammel	do	4	280	Poor	½	10
10	do	Freeport	Timber frame	4	310	Good	1½	40
11	do	Above Freeport	Brush and stone	2	150	Poor	¼	6
12	Cæsars Creek	Harveysburg	Plank and log	4	100	Poor	½	4
13	Little Miami River	Waynesville	Timber and stone	3½	150	Fair	1	15
14	Gladys Run	Spring Valley	Timber	4	35	Fair	¼	1
15	Little Miami River	Bellbrook	Log	3	150	Poor	1	15
16	Beaver Creek	Alpha	Stone fill	6	150	Fair	½	2
17	Little Miami River	Trebeins	Timber frame	9	230	Fair	1	25
18	do	Old Town	do	6	80	Good	½	5
19	Massicks Creek	Cedarville	Cut stone	24	30	Good	1	4
20	Little Miami River	Goes	Timber frame	6	90	Good	½	5
21	do	Above Goes	Timber and stone	4	75	Fair	½	4
22	do	South of Yellow Springs.	Timber	4	80	Fair	¼	3
23	Branch of Little Miami River.	Yellow Springs	Cut stone	10	50	Fair	¼	3
24	Little Miami River	Clifton	do	15	100	Good	½	10

of Little Miami basin.

Area of tributary watershed in square miles.	Use of dam, if water power; kind of mill.	Water power. Average head available.	Water power. Kind of wheels.	Water power. Total horsepower claimed.	Water power. Approximate time available.	Remarks.	Number.
					Mos.		
1,665	Flour mill	7	1 reaction, 2 turbines	25	10	Little used	1
353	Flour and feed mill	8	One 36-inch Leffel turbine.	25	8	Auxiliary steam.	2
245	Flour mill	20	Overshot, 20 feet diameter, 4 feet breast.	25	6	Little used	3
1,114	do	8	Two 48 and one 40 inch turbines.	40	10	do	4
1,106	Not used					Mill burnt	5
798	Flour mill	8	2 turbine	80	10		6
	Sawmill	8	One 20-inch turbine	30	10		
792	Powdermill	20	One 50 and two 56 inch Leffel turbines.	700	9	Auxiliary steam.	7
682	Flour mill	9	Two 48-inch Leffel and one 44-inch Victor turbines.	60	9		8
675	Feed mill	8	1 turbine	20	9	Little used	9
658	Flour mill	8	Three 36-inch Victor turbines.	100	8		10
	Sawmill	8	One 36-inch Victor turbine.	30	8		
653	Flour mill	7	40, 50, 60 inch turbines	35		Little used	11
201	Feed mill	8	One 40 and one 28 inch turbine.	40	8	do	12
410	Flour mill	8	2 turbines	75	8		13
	Sawmill	8	1 reaction	20	8		
22	Flour mill	24	1 American turbine	17	6	Auxiliary steam, 140 horsepower.	14
206	Sawmill	7	One 36-inch Victor turbine.	40	8	Little used	15
	Flour mill	7	One 40-inch Victor turbine.	50	8	do	
35	Feed mill	8	Overshot, 8 feet diameter, 15 feet breast.	25	6	do	16
246	Flour mill	11	One 48-inch American turbine.	40	8	Auxiliary gas	17
224	do	12	One 30-inch Victor turbine.	65	10	Auxiliary steam.	18
62	do	24	2 new American turbines, two 36-inch American turbines.	50	9	do	19
120	Powdermill	8½	One 30-inch Leffel turbine.	40	8	do	20
113	Flour and feed mill	9		40	9	Little used	21
109	Feed mill	6	One 48-inch Leffel turbine.	16	9		22
4	Not used						23
97	Flour mill	26	2 Leffel turbines, 26 and 15 inches.	50	9	Little used	24
	Sawmill	20	1 Leffel turbine, 26 inches	25	9	do	

In the Great Miami basin there are 53 dams, only 2 of which are not in use. Five divert water for the supply of the Miami and Erie Canal, and 48 furnish water power, 2 dams supplying both the canal and water power. The 48 dams furnish about 89 mills with a total horsepower of 4,158, or an average of 47 horsepower per mill. In 1880, according to the United States census, there were in the Great Miami basin 218 mills, using about 7,191 horsepower, or 33 horsepower per mill. In twenty years there was a decline of 59 per cent in the number of mills, but only of 42 per cent in the total horsepower in use.

In the Little Miami basin the table gives 24 dams, 2 of which are not in use, and the 22 remaining supply power for 28 mills, using 1,803 horsepower, or 64 horsepower per mill. In 1880 there were 59 mills, using 1,956 horsepower, which is 33 per mill. This is a decline of 52 per cent in the number of mills, but only 8 per cent in the total horsepower in use.

On the Miami and Erie Canal, within the Great Miami drainage basin, there are given 14 mills using 860 horsepower, or 61 per mill. In 1880 there were 48 mills using 2,011 horsepower from the canal. If the above figures are correct, there was a decline in twenty years of 70 per cent in the number of separate powers and of 57 per cent in the total horsepower in use.

The decline in the use of water power on the canal is no doubt mainly due to the falling off of navigation, which left many of the mills without sufficient means of obtaining supplies or of disposing of their product. The general tendency to use the more reliable steam power in preference to water power and the increase of competition were also factors in the abandonment of the small water powers.

In the water powers supplied from the streams direct the principal and almost only cause in a number of places was the decrease in run-off from the tributary drainage basin and the increase in the variation of this flow. This is a cause which has been expounded at such length that there is no further need for discussion here. It is altogether too true that the rapid deforesting of the State has seriously affected the regularity of the flow of the streams. Where there was formerly a fairly constant stream of water flowing there is now a variable one, dry in summer and with frequent floods in the spring.

The other factors which enter into the question of the decline in the use of water power are the filling up of the reservoirs and impounding areas and the natural deterioration of the plants.

The first of the above is one which can not be avoided and one which requires radical treatment to remedy the evil effects. With the greater variation in the flow of the streams and with a rapidly decreasing storage capacity the water power available was most seriously affected. Instead of being able to store the night flow for use during the day, it is necessary to depend upon the average daily flow of the stream.

The natural wearing out of the plants from long use, coupled with lack of means for repair and renewal, is causing many of the smaller plants to be abandoned. The old-school millers are dying off and the younger men, failing to find the operation of a small isolated mill remunerative enough, abandon it and seek other fields for employment.

The water power is employed in both these drainage areas for a much greater variety of work than in the other drainage areas. The isolated powers are still mainly used in flour, feed, and saw mills, but the power developed by the "hydraulics" in Dayton, Middletown, and Hamilton is used in a number of different industries. The large paper mills of the two last-named places derive

at least some of their power from this source. In a few cases where the water power is not sufficient, steam has been substituted and the water of the privilege used for boiler supply, washing purposes, etc.

The Hamilton "hydraulic" supplies some 12 mills and factories with 434 horsepower. The water is brought from the river by a race 5 miles long and is distributed to the various powers in such a way that some have the full head of 26 feet from race direct to the river, a few have a 13-foot head between the race and a tailrace, emptying into the old river bed, the remainder having 18 feet head from a second level to the river. The full head of this last group is not used at present, there being 7½ feet fall between the first and second levels.[a]

The next important power is found at Middletown, where the State dam sends a part of the flow of the river to a "hydraulic" or waterway, which now supplies 658 horsepower to 6 plants, including the city waterworks. The fall is obtained directly between the waterway and the river, and varies from 17 to 18 feet, according to the location of the power. This power is good for from ten to twelve months in the year and is one of the best in the district.

At Miamisburg about 330 horsepower is furnished to 3 mills by a dam in the Great Miami River at West Carrollton. In consideration of supplying a certain amount of water to the canal the waterway is used as a race for a portion of its length.

At Dayton a waterway from Mad River supplies 15 manufacturing establishments with 384 horsepower, not all of which is in use, however. The fall is obtained between different levels of the waterway, between these and the canal, and between the latter and the river. The water is not sufficient at all times for the power rented, and all of the latter is not used.

Above Dayton, at Harshmanville, Mad River furnishes about 300 horsepower to a number of small mills, but at the present time only 40 horsepower is used.

At Piqua a waterway, supplied principally from the canal, at one time furnished power to a large number of mills. At the present time, however, it furnishes only about 210 horsepower to the city waterworks.

In the Little Miami drainage basin by far the most important power is that developed at Kings Mills by the King Powder Company. At first the water was conducted through a long race to a number of isolated mills scattered along the river bank for 2 miles or more. Now it is conveyed to a central power plant, where it is used to generate electricity, which is in turn conveyed to the isolated mills necessary in the manufacture of powder.

MUSKINGUM BASIN.

As would be expected from its topography, this district was much more favorable to the development of water power than either of the other two.

The following table has been prepared to show the number and character of the dams still standing and the nature and amount of water power derived from them and also from the canal system:

[a] From Fifteenth Report Ohio State Board of Health, p. 569.

Dams and water power

Number.	Watershed and water course.	Approximate location of dam or water power.	Dam.				Length of backwater in miles.	Approximate pond area in acres.	Approximate area of tributary watershed in square miles.
			Character.	Condition.	Height in feet.	Length in feet.			
	MUSKINGUM RIVER.								
1	Muskingum River	Marietta	Timber frame	Good	12	480	5½	250	7,797
2	do	Devols	do	Good	10	588	8	500	7,783
3	do	Lowell	do	Good	11	848	10	900	7,736
4	Wolf Creek	Watertown	do	Fair	9½	90	1½	20	69
5	do	Wolf Creek	do	Poor	4	75	1	15	125
6	do	Pattons Mills	do	Poor	4	60	1	10	119
7	Muskingum River	Beverly	do	Good	10	705	8	600	7,473
8	do	Luke Chute	do	Good	11	546	6	350	6,981
9	do	Stockport	do	Good	11	482	9	500	6,915
10	do	McConnelsville	do	Good	11	472	8	400	6,849
11	do	Eagleport	do	Good	11	515	10	600	6,849
12	Salt Creek	Near Sundale	do	Poor	8	75	¼	2	4
13	Muskingum River	Taylorsville or Philo.	do	Good	9	736	9	700	6,646
14	Jonathan Creek	Powells	do	Poor	8	296	1	40	183
15	do	White Cottage	Cut stone	Good	9	108	1	20	158
16	Muskingum River	Zanesville	Timber frame	Good	8	514	10	600	5,850
17	Dresden Side Cut Canal.	Dresden							
18	Wakatomika Creek	Below Bladensburg	Log and stone	Poor	3	100	¼	2	28
19	do	Above Bladensburg	Log	Poor	5	75	¼	2	27
20	Ohio Canal	Adams Mills							
	LICKING CREEK.								
1	Licking Creek	Zanesville	Timber frame	Good	7	325	3	120	753
2	South Fork Licking Creek.	Kirkersville	do	Fair	1½	75	¼	1	48
3	North Fork Licking Creek.	Utica	do	Good	4½	170	1	20	117
	WILLS CREEK.								
1	Wills Creek	Plainfield	Timber frame	Poor	7	135	3	50	735
2	do	Linton Mills	Timber and stone.	Poor	8	150	4	70	708
3	do	Kimbolton	Timber	Fair	8	50	5	25	648
4	Salt Fork	Clio	Log and stone	Poor	8	50	4	20	83
5	Wills Creek	Below Cambridge	Log	Fair	7	60	4	20	464
6	do	Senecaville	Timber	Fair	5½	50	2	10	125

of Muskingum basin.

Water power.						
Use of dam or water power.	Head in feet.	Number and kind of water wheels.	Horsepower claimed.	Approximate time available (months per year).	Remarks.	Number.
Navigation, flour mill	12	Three 50, 52, and 66 inches	180	12		1
do	10	Three 48-inch turbines	80	12		2
Navigation, planing mill	13	One 42-inch turbine	85	12		3
Flour mill	13	One 48-inch turbine, one 54-inch reaction.	50	12		
Flour and feed mill	9½	One 30-inch turbine	45	9		4
Not used					Mill abandoned	5
Feed mill	4	Turbine	10		Very little used	6
Navigation, flour mill	10	Two 60-inch reaction	50	12		7
do	10	42 and 54 inch turbine	45	12		
Sawmill	10	One 48-inch turbine	55	12		
Planing mill	10	One 58-inch reaction	35	12		
Navigation						8
Navigation, flour mill	11	72, 72, and 60 inch reaction	50	12		9
do	11	Six 54-inch reaction, two 60-inch reaction.	120	12		10
Navigation						11
Feed and saw mill	8	3 turbines, 35, 28, and 24 inches.	30	6		12
Navigation, flour mill	9	One 60-inch turbine, two 54-inch turbines, three 48-inch turbines.	135	12		13
Feed and flour mill	8	6 reaction	20		Little used	14
do	9	One 42-inch turbine, three 48-inch reaction.	40	9		15
Navigation, flour mill	8	Two 60-inch turbines	50	12		16
Coffin factory	13	One 35-inch turbine	55	12		
Furniture factory	14	do	60	12		
Ice factory	16	do	50		Little used	
Woolen mill	22	One 22-foot overshot, 6-foot breast.	25	7	Canal to river	17
Feed mill	12	Turbines	15	8	Little used	18
do	6	do	10	8	do	19
Flour mill	22	One 36-inch turbine	96	12	Canal lock	20
Flour mill	7	Two 40-inch turbines	100	12		1
Canal feeder, saw and feed mill.	11	Turbine	30		Little used	2
Flour mill	9½	Two 35-inch turbines	30	6		3
Not used					Mill abandoned	1
Feed mill	8	1 turbine, 1 reaction	30		Little used	2
Flour and saw mill	8	Turbines	25			3
Not used					Mill standing	4
Saw and flour mill	7	48-inch turbine	37	8		5
Flour mill	5½	One 48-inch turbine	20	6		6

Dams and water power of

Number.	Watershed and water course.	Approximate location of dam or water power.	Dam.				Length of backwater in miles.	Approximate pond area in acres.	Approximate area of tributary watershed in square miles.
			Character.	Condition.	Height in feet.	Length in feet.			
	WALHONDING CREEK.								
1	Walhonding Feeder.	Roscoe							
2	Walhonding Creek	Above Coshocton	Timber	Good	4	275	½	15	2,148
3	Deughty Fork	Below Clarks	Log	Fair	5	80	¼	1	46
4	do	Above Clarks	Stone and timber frame.	Fair	6½	150	½	15	31
5	do	Becks Mills	Timber frame	Fair	9	100	¼	1	15
6	do	East of Saltillo	do	Fair	14	100	1	10	12
7	do	Northeast of Saltillo.	Stone and timber.	Fair	11	80	⅛	1	10
8	do	3 miles south of Berlin.	Timber frame	Poor	14	100	½	8	8
9	Killbuck Creek	Killbuck	Timber	Fair	6	50	2	10	421
10	Point Creek	Point Valley	do	Poor	5	25	½	1	16
11	Salt Creek	Fredericksburg	do	Fair	3	25	½	3	11
12	Run to Killbuck	Near Mill Brook	Earthen	Poor			½	1	1
13	Apple Creek	East of Wooster	Timber frame	Fair	6	30	¼	1	29
14	Apple Creek, North Branch.	Northeast of Wooster.	do	Fair	4	30	¼	2	12
15	Killbuck Creek	Burbank	do	Poor	18	100	½	3	42
16	Beaver Run	Shannon	Earthen	Fair	7	300	⅛	3	2
17	Kokosing River	Millwood	Log	Fair	4½	200	½	7	449
18	Big Schench Creek	Monroeville	Timber	Fair	5	75	½	1	26
19	Kokosing River	Gambier	Timber frame	Good	5	233	½	20	304
20	do	Mount Vernon	do	Good	6	150	1	12	188
21	do	Fredericktown	Log and gravel	Poor	7	75	½	2	71
22	do	Above Fredericktown.	Timber frame	Fair	4½	115	½	3	39
23	do	Ankenytown	Timber	Poor	3	65	½	1	20
24	Mohican River	Brinkhaven	do	Good	5½	270	1½	50	949
25	Run to Jerome Fork	Above McZena	Earthen	Poor					1
26	do	Mohicanville	Timber frame	Fair	11	30	½	1	3
27	do	Southeast of Springville.	do	Fair	10	20	½	1	2
28	do	Springville	Earthen	Poor				1	1
29	Jerome Fork	Jeromeville	Timber	Fair	8	45	½	1	123
30	Clear Fork	Below Butler	Stone	Poor	2	50	⅛	1	130
31	do	Above Butler	Timber frame	Fair	3½	75	½	2	133

Muskingum basin—Continued.

Use of dam or water power.	Water power. Head in feet.	Number and kind of water wheels.	Horsepower claimed.	Approximate time available (months per year).	Remarks.	Number.
Flour mill	14	One 36-inch turbine	50	12	Canal to creek	1
Sawmill	12	Turbine	25	12	do	
Flour mill	22	One 22-foot overshot, one 28½-inch turbine.	93	12	Feeder to canal	
Woolen mill	18	One 32½-inch turbine	25	12	do	
Canal feeder						2
Flour mill	8½	One 40-inch turbine	20	8		3
Sawmill	7	One 30-inch turbine	12	7	Little used	4
Woolen mill	9	One 18-inch turbine	8	7	do	
Sawmill	8	One 30-inch turbine	20	6	do	5
Flour and feed mill	14	14-foot overshot, 4-foot breast.	10	6	do	
Woolen mill	16	One 12-inch turbine	6		do	6
Sawmill	12	One 48-inch turbine	25		do	7
Flour and feed mill	15	One 22-inch turbine	37	6		8
Flour and saw mill	6	3 turbines, 3 reaction	80	9		9
Saw, cider, and feed mill	20	One 13-inch turbine	10	6		10
Woolen mill	14	14-foot overshot, 3-foot breast.	12			11
Feed and cider mill			12		Very little used	12
Flour and feed mill	12	Turbine	25	10		13
Cider mill	8		10		Very little used	14
Not used					Mill burned	15
Feed, cider, and saw mill	18½	18½-foot overshot, 4-foot breast.	12	6		16
Flour mill	5	2 turbines, 35 and 48 inches	25	12		17
do	18	One 18-inch turbine	35		Little used	18
do	12	2 turbines, 40 and 26½ inches	80	12		19
do	12	Turbines	115	8	50 horsepower year around.	20
do	7½	One 32-inch turbine	32	8		21
Sawmill and cider press	15	Turbine	20		Little used	22
Feed mill	12	One 20-inch turbine	10	4		23
Flour mill	8	3 turbines, 40, 44, and 50 inches.	130	12	Only 80 horsepower used.	24
Saw and cider mill	10		6		Little used	25
Flour and feed mill	22	22-foot overshot, 4½-foot breast.	20	8		26
Saw and cider mill	20	20-foot overshot, 1½-foot breast.	24	12		27
Feed mill	10	Turbine	10		Little used	28
Flour and feed mill	8	do	40	6		29
Saw and flour mill	10	Reaction and turbine	57		Little used	30
Flour and feed mill	10	10-foot overshot, 16-foot breast.	25			31

Dams and water power of

Number.	Watershed and water course.	Approximate location of dam or water power.	Dam.						Approximate area of tributary watershed in square miles.
			Character.	Condition.	Height in feet.	Length in feet.	Length of backwater in miles.	Approximate pond area in acres.	
	WALHONDING CREEK—continued.								
32	Clear Fork	Bellville	Stone and brush.	Poor	2	70	¼	1	112
33	do	3 miles above Bellville.	Log and frame.	Poor	2	75	¼	1	60
34	Black Fork	Above Loudonville	Timber frame	Good	5	330	2	50	340
35	do	Perrysville	Log and stone	Poor	8	150	3	10	316
36	Run to Rocky Fork	Lucas	Earth and frame.	Poor	3	100	¼	2	4
37	Rocky Fork	Below Mansfield	Timber	Poor	8	60	¼	2	42
	TUSCARAWAS RIVER.								
1	White Eyes Creek	Avondale	Stone fill	Poor	6	42	¼	1	36
2	Ohio Canal	New Comerstown							
3	do	Port Washington							
4	do	Lock Seventeen							
5	Tuscarawas River	West of Uhrichsville	Timber frame	Fair	5	400	1¼	70	2,237
6	Big Stillwater	Above Uhrichsville	Log	Poor	2	100	1	10	355
7	do	Tippecanoe	Cut stone	Good	10	125	5	70	193
8	Tuscarawas River	South of Midvale	Loose stone	Poor	2	300	¼	10	1,760
9	Ohio Canal	Blakes Mills							
10	Tuscarawas River	New Philadelphia	Stone fill	Fair	6	250	2	50	1,656
11	Sugar Creek	Canal Dover	Timber frame	Poor	5	250	1	10	345
12	do	Strasburg	Loose stone	Poor	3½	175	¼	5	313
13	South Fork Sugar Creek.	Dundee	do	Fair	7	70	¼	2	66
14	do	Barrs Mills	do	Fair	6	40	¼	1	59
15	One Leg Creek	Sherodsville	Log	Poor	7	75	2	10	142
16	Tuscarawas River	Zoar	Timber	Poor	5	150	2	15	1,016
17	Nimishillen Creek	Sparta	do	Fair	6	75	1	6	166
18	do	Howenstine	Log	Poor	4	150	¼	2	158
19	do	Below Canton	Timber frame	Fair	5	100	¼	5	134
20	Nimishillen Creek, West Branch.	Above Canton	do	Good	1½	30	¼	1	43
21	Sandy Creek	Magnolia	do	Good	5	208	1½	30	251
22	do	Malvern	do	Good	8	400	1	40	135
23	do	Oneida	Log	Good	6	100			133
24	Wolf Creek	North of Loyal Oak	do	Poor	6	20	¼	1	30
25	Tuscarawas River	Millheim	Earth and frame.	Fair	7	35	¼	5	15

Muskingum basin—Continued.

Water power.						
Use of dam or water power.	Head in feet.	Number and kind of water wheels.	Horsepower claimed.	Approximate time available (months per year).	Remarks.	Number.
Flour and feed mill	7½	Turbine	25	8		32
Feed mill	6	....do	10		Little used	33
Flour mill	8	....do	75	6		34
Not used						35
....do						36
Feed mill	8	Turbine	15		Little used	37
Feed mill	6	One 40-inch turbine	15		Little used	1
Flour mill	7½	2 turbines, 36 and 40 inches	50	12	Canal lock	2
....do	6½	One 48-inch turbine	80	12	....do	3
....do	9	One 35-inch turbine	45	12	....do	4
Canal feeder, flour mill						5
Not used					Mill abandoned	6
Flour mill	10	1 new turbine, 2 old turbines	30	9	Just remodeled	7
Not used					Mill abandoned	8
Flour mill	11	2 turbines	70	12	Canal lock	9
Electric power house	10	Two 56-inch turbines, one 45-inch turbine.	300	8		10
Canal feeder, flour mill	14	Turbines	75		Feeder to canal	11
Flour mill	7	....do	50		Canal to river	
Sawmill	12	One 44-inch turbine	30	12		12
Flour and feed mill	7	Turbine	11	6		13
Flour, feed, and cider mill	6	....do	20	8		14
Saw and feed mill	7	....do	15		Little used	15
Canal feeder, saw and flour mill.	5	5 turbines	80	12		16
Flour mill	19	3 turbines	50	10		17
Sawmill	4		10		Little used	18
Flour mill	9	5 turbines	40			19
Canton water supply						20
Flour mill	10½	Turbine, two 10½-foot overshots, 12-foot face.	80	10		21
....do	15	Turbines	25	12		22
Flour mill and railroad water.					Under construction.	23
Feed mill	6		10		Little used	24
....do	10	Overshot	15		....do	25

The information contained in the above table was secured in nearly every case from the mill owners. The data in reference to the backwater could be obtained by actual measurement in but few cases, and much of it is therefore only approximate. The areas given for the tributary watersheds were obtained by planimeter measurements.

The table shows 90 water powers, as against 253 reported by Prof. Dwight Porter in the Tenth Census report as in existence in 1880. Owing to the large area covered it is probable that a few unimportant powers have been overlooked, but their number can not be great, as special inquiry was made on this subject. Considering the large number of powers that have been abandoned in the last ten years, this reduction of 65 per cent is not hard to believe. Throughout this district there are streams which only a few years ago supported seven or eight powers and which now have but one or two. This decline in the number of water powers is brought out in another light by the following: In 1880 the total horsepower given was 7,066, while in the above table there is listed 3,960, a reduction of 44 per cent. This is due to some extent to the fact that the small powers escaped notice. These powers, located on small creeks, have been first to suffer from the lack of uniformity in the flow of the streams. Many of the creeks that formerly maintained a good flow throughout the year are now dry in summer and fall, and at flood height during short periods in winter and spring, thus leaving but a few months in the latter seasons during which power could be obtained.

There are 84 dams described, 8 of which are not in use at all; 10 were built for the slack-water navigation of the Muskingum, 4 for canal feeders, and the remaining 62 were put in for power purposes alone. There is a small dam in Tuscarawas River near its headwaters which diverts water to the canal reservoirs and feeders, and there is a waste dam in the same stream where the canal crosses it above Clinton, which, together with several small temporary dams, have not been given in the table.

All the dams given in the table as in poor condition and the mills marked as being little used have not been repaired for years, and the next decade will probably see them abandoned if present conditions hold.

The greatest power in the drainage area is obtained from the canal system and from the slack-water navigation system in Muskingum River. Those at Adams Mills, New Comerstown, Port Washington, Lock Seventeen, and Blakes Mills are on the Ohio Canal proper and utilize the fall at a lock, or, in the case of Adams Mills, at a double lock. The water supply for these mills is usually sufficient to operate the wheels at full gate. In fact, the decline of navigation has in a way increased the amount of water available at the mills, as a smaller quantity is required in the operation of locks. At times during the winter ice blocks the canal and cuts off the supply for a short time, but the water seldom fails in dry seasons.

The mill at Dresden is located on the "Dresden side cut," and obtains its head from the fall between the canal and Muskingum River. This side cut carries off much of the surplus water of the canal between the Licking and the Portage summits, and an excellent power is available for development.

Two of the mills at Roscoe use the water from the Walhonding feeder as it passes to the canal proper, and two take water from the canal and conduct it to the creek. These powers are of great value, and much more could be obtained from them by a modern installation. The power on the Kirkersville feeder is of but slight importance and can be developed but little, as the tributary drainage area is too small to secure a steady supply.

Sugar Creek at Canal Dover should furnish an excellent water power, but the works have been neglected and are now in poor condition. The water is used both as it goes from the feeder to the canal and from the canal to the river. These water privileges were granted to the owners in payment for the diversion of Sugar Creek for canal purposes.

At the old town of Zoar there is a combination of water powers, listed as one in the table, which use water from the Zoar feeder dam and conduct it back to the river. These were also granted in payment to former riparian owners for the diversion of part of the flow of the Tuscarawas to the canal.

The fourteen water powers now in use at the eight Government dams, as shown in the table, were granted when the supply of water was plentiful, for the purpose of promoting industries along the river. Thus the amount of navigation on the slack-water system, which was then not under Government control, was increased. Now the amount of water is not sufficient for both navigation and power, and the latter has become subordinate to the former. Many of the leases were granted for long periods and at low rates and have become very valuable. The one at McConnellsville, given in perpetuity as payment for a diverted water right, guarantees to the present owners, free of cost, a supply of about 17,000 cubic feet of water per minute at a head of 11 feet. The lease provides for "enough water for 10 run of 4-foot 5-inch stones."

Of the powers derived from the dams built for this purpose alone the best are those at Zanesville, on Licking Creek; at Killbuck, on Killbuck Creek; at Gambier and Mount Vernon, on Kokosing River; at Brinkhaven, on the Mohican; at Magnolia, on Sandy Creek, and at New Philadelphia, on Tuscarawas River. The last-named plant is provided with modern wheels, well installed, and is a good though variable power. An important cause of interruption at this power site is backwater during freshets and floods. The dry-season flow is usually sufficient.

As an indication of the generally low estimate of the value of these water powers, it may be well to state that by conversation with a

large majority of the mill owners it was found that the powers were for sale at a low figure. The future may find many of these plants offering a profitable field for the investment of capital.

PUBLIC WATER SUPPLIES.

NUMBER AND EXTENT.

Within the drainage areas here described 104 cities and villages, having a total population of 771,128, are provided with public water-supply systems. In the Sandusky basin are 8, comprising a population of 35,988; in the Maumee are 5, with a population of 287,633; in the Miami are 34, with a population of 246,499, and in the Muskingum 37, with 201,008 inhabitants. Eighteen of these towns, with a population of 11,627, are each below 1,000 and are not classed as urban, thus leaving 86 cities and urban towns, with a total population of 759,501, having public water-supply systems. These towns represent 61 per cent of the total number of urban towns and 91 per cent of the urban population.

In the Sandusky drainage area there are no urban towns without water supplies, while the Maumee has 22, with a total population of 35,562. In the Great Miami basin there are 10, with a population of 16,314, and in the Little Miami 8, with a total population of 28,199. In the Muskingum are 6, with a population of 8,169, only 6,969 of which, however, is within the drainage area, the corporate limits of some extending over into other basins.

In a few communities, by prohibiting the construction of vaults and requiring the use of boxes or dry earth closets, and especially by compelling these to be properly cleaned, the soil and subsoil have been kept clean, and potable water is provided from wells.

In a few instances it has been found that the private well supply is obtained from a stratum whose water is but little influenced by surface pollution. That these cases are not more numerous is probably due to the great expense usually attached to obtaining the deep-seated water. Nearly every village has one or more wells which reach desirable water, but only a favored few can afford these, and the majority must use the more easily obtained but less potable subsurface supply.

Five of the public water-supply systems of the Maumee basin are in Indiana and Michigan, which leaves for that area in Ohio 20 water-works in towns having a total population of 224,260. No recent information is available except for the Ohio water supplies, and the details of these only will be given.

Following is a list of the water-supply systems, grouped according to the drainage areas, showing the name of the city or town, its estimated population, and the source of supply.

Source of water supply of cities and towns in Sandusky, Maumee, Miami, and Muskingum drainage basins.

SANDUSKY BASIN.

City or town.	Population.	Source of supply.
Attica	694	Wells and galleries.
Bucyrus	6,560	Sandusky River.
Carey	1,816	Wells.
Crestline	3,282	Wells and springs.
Fremont	8,489	Sandusky River, wells, and galleries.
Sycamore	853	Wells.
Tiffin	10,989	Sandusky River and wells.
Upper Sandusky	3,355	Sandusky River.
Total	35,988	

MAUMEE BASIN.

Ada	2,576	Wells.
Bluffton	1,783	Do.
Bryan	3,131	Do.
Columbus Grove	1,935	Do.
Continental	1,104	Do.
Defiance	7,579	Maumee River.
Delphos	4,517	Wells.
Findlay	17,613	Blanchard River.
Hicksville	2,520	Wells.
Lima	21,723	Lost Creek and wells.
Montpelier	1,869	Wells.
Napoleon	3,639	Maumee River.
New Bremen	1,318	Wells.
Paulding	2,080	Do.
Rockford	1,207	Do.
St. Marys	5,359	Do.
Toledo	131,822	Maumee River.
Van Wert	6,422	Wells.
Wapakoneta	3,915	Do.
Wauseon	2,148	Do.
Total	224,260	

Source of water supply of cities and towns in Sandusky, Maumee, Miami, and Muskingum drainage basins—Continued.

MIAMI BASIN.

City or town.	Population.	Source of supply.
Batavia	1,029	East branch of Little Miami.
Bellefontaine	6,649	Wells.
Blanchester	1,788	Impounding reservoir and wells.
Dayton	85,333	Wells.
Eaton	3,155	Gallery and wells.
Evanston	1,716	Ohio River.
Franklin	2,724	Wells.
Greenville	5,501	Do.
Hamilton	23,914	Do.
Harrison	1,456	Do.
Hyde Park	1,691	Ohio River.
Lebanon	2,867	Wells.
Lynchburg	907	Do.
Madisonville	3,140	Do.
Middletown	9,215	Do.
New Vienna	805	Do.
Norwood	6,480	Do.
Oakley	528	Ohio River.
Oakwood	342	Do.
Osborn	948	Wells.
Oxford	2,009	Do.
Piqua	12,172	Miami and Erie Canal.
Pleasant Ridge	953	Ohio River.
Sidney	5,688	Wells.
Springfield	38,253	Gallery and Buck Creek.
Tippecanoe City	1,703	Wells.
Trotwood	220	Do.
Troy	5,881	Do.
Urbana	6,808	Do.
Versailles	1,478	Do.
Waynesville	723	Do.
West Alexandria	740	Do.
West Carrollton	987	Do.
Xenia	8,696	Impounding reservoir, springs, surface-water, and wells.
Total	246,499	

Source of water supply of cities and towns in Sandusky, Maumee, Miami, and Muskingum drainage basins—Continued.

MUSKINGUM BASIN.

City or town.	Population.	Source of supply.
Ashland	4,087	Wells.
Barberton	4,354	Do.
Canal Dover	5,422	Do.
Canton	30,667	Nimishillen Creek and wells.
Cambridge	8,241	Wills Creek.
Carrollton	1,271	Wells.
Coshocton	6,478	Do.
Dalton	666	Well.
Dennison	3,763	Big Stillwater Creek.
Freeport	690	Wells.
Granville	1,425	Do.
Lewisville	170	Do.
Loudonville	1,581	Do.
Mansfield	17,640	Do.
Massillon	11,944	Slipper Creek and wells.
Marietta	13,348	Ohio River.
McConnelsville	1,825	Wells.
Millersburg	1,998	Do.
Minerva	1,200	Do.
Mount Vernon	6,633	Do.
Mineral City	1,899	Springs.
Mineral Point	1,220	Do.
Newark	18,157	Licking Creek and wells.
New Comerstown	2,659	Wells.
New Philadelphia	6,213	Do.
Orrville	1,901	Do.
Perrysville	513	Do.
Piedmont	275	Springs.
Scio	1,214	Wells.
Shreve	1,043	Well.
Shelby	4,685	Wells.
Uhrichsville	4,582	Big Stillwater Creek.
Wadsworth	1,764	Springs and wells.
Waynesburg	613	Springs.
Wooster	6,063	Christmas Run, Apple Creek, and wells.
Zanesville	23,538	Muskingum River.
Total	201,008	
Grand total	771,128	

Dennison and Uhrichsville have the same supply, furnished by a private water company, so that in the Muskingum drainage basin 36 waterworks supply 37 cities and villages.

CLASSIFICATION.

For convenience in discussing the various supplies, they have been divided, according to the source from which the water is obtained. The divisions made are based upon the origin of the water, whether from surface or ground. A second division classifies the ground water as subsurface or deep-seated. The subsurface class include all ground waters found in porous beds near the surface and of recent surface origin, while the deep-seated class includes those waters found in deeper strata and of a more remote surface origin; in other words, in the first class are put all waters the purity of which is easily affected by surface pollution, and in the second class those waters which are so far removed from direct connection with the surface that they are not easily influenced by surface pollution.

All the large towns are supplied either entirely or in part with surface water. It has seemed almost impossible, with a reasonable expenditure, to procure in these drainage areas sufficient ground water for even the larger villages. The only city of any size in the "ground" division is Mansfield, which, by the expenditure of an enormous amount of money, has been able to procure enough ground water for the supply of the city. Mansfield has the second most expensive plant of the 104 named, being surpassed by Toledo only, a city with six times the population.

None of the surface-water supplies are fit for domestic use unless they are artificially purified, as in the case of the supply for Dennison and Uhrichsville. Ten towns of the 16 supplied with surface water are provided with water that is so objectionable that only for short periods can it be considered at all fit for drinking purposes. This condition is generally recognized by the inhabitants of these towns, and better water is being sought for in several of these places. Bucyrus is now impounding a new surface supply from a small drainage area, Cambridge is seeking ground water, Findlay and Toledo are agitating the question of better supplies, and Zanesville is investigating the relative cost of filtering the water it now gets or of obtaining a ground water.

In the last main division, "Surface and ground supplies," there are 10 towns—Blanchester, Lima, Springfield, Wooster, Xenia, Canton, Massillon, Tiffin, Fremont, and Crestline—which were originally supplied with surface water. Owing to the objectionable character of the water, they have tried to procure ground water, but failed to obtain enough to allow them to abandon their surface supplies. In justice to Massillon it must be stated that all the water for domestic

use is obtained from wells and is of excellent quality, and that the surface water is retained for industrial use only.

Newark started with ground water, but, owing to the insufficiency of the supply, was compelled to use some surface water, and with little detriment, as the water is of good quality.

The small villages located in the glaciated area of the State usually have no difficulty in obtaining plenty of water either from the drift or from the rock under it. This rock water is of course available in the unglaciated portion also, but, unfortunately, in this section it is frequently so highly impregnated with mineral matter as to be objectionable.

In many localities in the Maumee drainage basin flowing wells can be obtained, which usually furnish water that is excellent for domestic use, but not so desirable for industrial purposes, because of dissolved incrusting salts. This region of flowing wells, or "fountains," as they are locally called, extends up into Michigan, where several towns obtain public supplies from wells of this kind.

DETAILED DESCRIPTION OF WATERWORKS.[a]

Following are detailed descriptions of the public water supplies of the four drainage areas.

SURFACE SUPPLIES.

Batavia.—A public water-supply system was installed in 1900 by Batavia (population, 1,029), and has cost to date $38,000, including the electric-light plant. The supply is obtained from the East Branch of Little Miami River, at a point just above the village and above all local pollution. As may be seen from the description of the drainage basin, the East Branch drains a hilly farming country, and receives no notable pollution until Lynchburg, 42 miles above, is reached, at which place a large distillery winters some 1,000 head of cattle, the filth from which directly enters the stream. Before this source was approved filtration was required by the State board of health. The village first presented plans for a rather crude system of slow sand filtration, which were here hardly satisfactory, and finally plans for mechanical filtration by the Wefugo process, which were approved. The water is pumped from the river by a low-service pump to two sedimentation tanks situated on the side hill back of the filter house and pumping station. From these tanks the water flows by gravity onto the filter, and through it to a clear-water well, from which it is pumped to a supply reservoir situated on the high ground north of the village.

The filtration process involves the use of a coagulant—sulphate of aluminum in this case—which is admitted to the water as the latter passes to the sedimentation basins. The coagulant solution is pumped

[a] Many of the facts reported under this head have been published by the Ohio State board of health.

from the coagulant tanks by a small piston pump actuated by the low-service pumps, so that a constant ratio is maintained between the amount of solution and the amount of water pumped. The actual amount of coagulant used must be varied by changing the strength of the solution; it is usually 1 grain per gallon. At times of high turbidity in the stream it is necessary to increase this amount in order to get perfect clarification.

The sedimentation tanks are two in number and consist of open wooden tubs, each 20 feet in diameter and 13½ feet high, with an effective height of 10 feet, giving a capacity of 23,660 gallons each, or 47,320 gallons for both. These tanks are exposed to the weather, with no protection whatever. As the water enters the tanks it falls over the sides of a pan secured to the discharge pipe of the pump, and receives more or less aeration. The tanks are used alternately, an automatic device admitting the water to first one and then the other, thus securing a longer period of quiet sedimentation. The outlet from these tanks is supplied with a skimmer float, so as to procure at all times the clearest water for the filters. The tanks are supplied with suitable pipes for flushing out the collected sediment.

The filter consists of an 8-foot cypress tank, 8 feet deep, fitted with a double bottom, the upper bottom being fitted with brass strainers supplied with phosphor-bronze screens. The openings in the screens are approximately three-fourths by one one-hundredth of an inch. In the bottom of the strainer is placed a brass ball, so arranged as to throttle the downward flow but to allow a free upward flow of the wash water. On the screens rests a 2-foot 9-inch bed of sand, composed of crushed quartz. The compartment between the bottoms is for the collection of the clear water, and to this is connected the discharge pipe of the filter, which is fitted with a regulator, so that the rate of filtration is kept uniform, irrespective of the condition of the bed. The inlet pipe is controlled by means of a float, so that a uniform level is maintained on the bed. A positive head of 4 feet is maintained on the filter and a suction head of 7 feet, making a total of 11 feet.

The filter is washed by reversing the flow through the sand and at the same time stirring it with a revolving mechanical agitator. The wash water is carried off by an annular trough fastened inside of the filter tank.

The total filtering area is 50 square feet, or 0.0012 of an acre, giving, at the standard capacity of 120,000,000 gallons a day, a rate of filtration of 100,000,000 gallons a day per acre.

The clear well is 42 feet in diameter and 16 feet deep, with a capacity of 165,000 gallons.

The low-service pump is of the horizontal duplex McGowan type, of 500,000 gallons capacity. The water is drawn from the clear well to the reservoir by two horizontal duplex McGowan pumps of 750,000 gallons capacity each. The pressure at the station is 95 pounds per

square inch, and the average on the mains is somewhat less than this. The reservoir is 30 feet in diameter and 17 feet deep, its capacity being 84,000 gallons. Both this reservoir and the clear well have brick walls laid in cement and concrete bottoms, but neither is covered. There are 3½ miles of mains, which serve about 95 per cent of the people. There are 140 service taps, all of which are in use and none metered. The average daily consumption is 25,000 gallons for the total population, equal to 24 gallons per capita of the population, or 25 gallons per consumer. The water after filtration is clear and seems to be of excellent quality, and should displace the private wells, many of which are badly contaminated.

Bucyrus.—The public water-supply system of Bucyrus (population, 6,560) was established by a private company in 1883 and has cost to date about $125,000. The supply is taken from Sandusky River and forced through mechanical filters directly to the mains. Owing to the small amount of water flowing in the river in the summer, it is necessary to impound some to tide over this season. This is done by three small dams across the river, one 1½ miles above Bucyrus, one just above the city, and one at the pumping station. The dam at the station is constructed of timber and is 70 feet long, 2½ feet high, and impounds about 2,000,000 gallons of water. The one above this is a temporary structure of brush and stone, but is so favorably located as to hold back nearly 2,000,000 gallons of water. The third dam is constructed of timber, is 30 feet long, 2½ feet high, and impounds a large expanse of water, as the river has but little fall above. This dam flooded so much land that the water company has been compelled by the courts to abandon it.

The water flows from the backwater of the dam at the station into a large covered well, which serves as a pump well, but which was first put in as an infiltration basin in a vain attempt to furnish ground water to the city. This well is constructed of brick, is 30 feet deep, and 50 feet in diameter.

The water is pumped from this by a compound duplex Holly pump of 1,500,000 gallons capacity and a compound duplex Walker pump having a capacity of 1,000,000 gallons. Before reaching the consumers the water goes through two Hyatt pressure filters, which were erected in 1886. They are the old-style vertical filter, 10 feet in diameter, and contain 2½ feet of crushed granite and 1½ feet of coke. They are washed by reversing the flow through them once or twice a day, as the condition of the water may require. Alum is used as a coagulant when the water is very muddy, but not continuously enough or in quantities large enough to be constantly effective. The filters serve merely as strainers and give no protection from the dangerous ingredients of the water.

Sandusky River is badly polluted with the sewage of Crestline, only 16 miles above Bucyrus. This fact has led the company to seek

a new supply. A large impounding reservoir 2½ miles up the river, east of the city, has just been constructed, which holds the run-off of a small tributary to the river. This tributary has a drainage area of only about 3 square miles, so that a large reservoir was necessary in order to impound sufficient water. This reservoir was formed by building across the lower end of the run an earthen embankment with a masonry waste weir, the embankment being made of clay and soil stripped from the area to be flooded. The reservoir has an area of about 60 acres and holds 166,000,000 gallons of water, which stands at an average depth of 8 feet, though over much of the area it is very shallow. The water flows by gravity through a 16-inch pipe to the present pumping station and is forced directly to the mains, the filters being abandoned.

About 12 miles of mains have been laid, which supply water to about 70 per cent of the people. There are 500 service taps connected, of which 325 are in use and 175 are metered. The average daily quantity of water pumped is 622,000 gallons, but owing to the waste at the filters not more than 550,000 gallons is sent to the mains. This is 65 gallons per capita for the total population, or 280 gallons for each consumer.

Cambridge.—A system of public water supply was established by Cambridge (population, 8,241) in 1896 and has cost to date about $63,000. The supply is taken directly from Wills Creek, which is a very sluggish, muddy stream, draining a hilly mining country. Its waters are polluted by filthy surface drainage from mining communities and by the drainage from the mines themselves. The water is so objectionable that it is not employed for domestic purposes, for which private wells are used. A ground-water supply is now being developed, which, if sufficient in amount, will be preferable.

The water is pumped from the creek to a standpipe by two horizontal compound duplex Laidlaw-Dunn-Gordon pumps of 1,500,000 gallons capacity each. The standpipe is located on a hill between the pumping station and the city, and when full gives an average pressure of 70 pounds per square inch in the mains. It is 60 feet high, 25 feet in diameter, and has a capacity of 200,000 gallons.

There are 12 miles of mains, to which 70 per cent of the people have access. There are 400 services connected, of which 140 are in use and 130 are metered. The large number of idle services is due to the fact that a number of streets have just been paved, and all the services have been put in to the curb line to avoid tearing up the new pavement. The average daily consumption is claimed to be 400,000 gallons, which is 57 gallons per capita of the city and 440 gallons per capita of the population actually supplied. This large figure is due to the general waste of the water and to its extensive use for industrial purposes.

Defiance.—A system of public water supply was installed at

Defiance (population, 7,579) by a private company in 1887 and had cost to August, 1898, about $200,000.

The supply is taken directly from the Maumee River at a point above all local contamination from sewage, but within the influence of that of Fort Wayne, only 54 miles above. The water is pumped from the river to a standpipe by two horizontal compound duplex Blake pumps, each of 2,000,000 gallons capacity. The standpipe is 140 feet high and 20 feet in diameter and has a capacity of 329,000 gallons. It furnishes an average pressure on the mains of 55 pounds per square inch. Eighteen miles of mains have been laid, which supply about 75 per cent of the population with water. Seven hundred services have been connected, of which 500 are in use, 40 of which are metered. The average daily consumption is about 1,100,000 gallons, or 145 gallons per capita of the total population and 193 gallons per capita of the population actually supplied.

The water is not used for domestic purposes because of its muddiness during six months of the year.

Dennison, Uhrichsville.—The twin cities of Dennison (population, 3,763) and Uhrichsville (population, 4,582) are supplied with water from a plant put in by the Dennison Water Supply Company in 1888. The cost to October 31, 1902, was $312,121.45.

Until recently the supply was taken directly from Big Stillwater Creek and with but a slight preliminary sedimentation was pumped to a small distributing reservoir. Big Stillwater Creek is an exceedingly sluggish and turbid stream, carrying much objectionable surface and mine drainage, and the water taken from it was almost unusable for domestic purposes. In January, 1900, the water company put in a mechanical filter plant, and an excellent supply has since been furnished the city. The raw water is admitted from Stillwater Creek to a settling basin of about 400,000 gallons capacity, flows from here to a small suction well, where the coagulant is applied, and is then pumped onto the filters by a horizontal duplex Worthington pump of 1,000,000 gallons nominal capacity, but of greater capacity now on account of the low head. There are four standard 16-foot Jewell gravity filters, each of which has a filter bed 15 feet in diameter and 4½ feet deep, composed of natural sand from Red Wing, Minn. The area of each filter is 176.7 square feet, making 706.8 square feet, or 0.016 acre for the four filters, which will give them a combined capacity of 1,920,000 gallons per day. The coagulant used is crushed block alum or aluminum sulphate. The average amount used is 0.75 grain per gallon, which amount is sufficient to produce clear effluent.

From the filters the water flows through a concrete flume to a clear-water well, from which it is pumped to the distributing reservoir by a horizontal compound duplex Holly pump of 2,000,000 gallons capacity. The reservoir is located on a hill between the pumping station and Dennison and gives an average pressure of 100 pounds. It is an open

reservoir, its sides being lined with brick and its bottom puddled with clay, and has a capacity of 3,000,000 gallons.

There are about 17 miles of mains in both cities, supplying with water 95 per cent of the inhabitants of Dennison and 75 per cent of those of Uhrichsville. Altogether there are 1,235 service taps in use, only one of which is metered. The average daily consumption is 1,700,000 gallons, or 170 gallons per capita of the total population, and 422 gallons per capita of the population actually supplied. Railroad shops and tanks consume 50 per cent of this, other manufacturing and commercial plants about 10 per cent, leaving for domestic use about 68 gallons per capita daily.

Evanston, Hyde Park, Oakley.—Cincinnati supplies with Ohio River water the suburbs of Evanston (population, 1,716), Hyde Park (population, 1,691), and Oakley (population, 528). They are supplied as a portion of the city proper, with a slight modification of rates, compensation for fire protection, etc. No separate data are available at this time as to the extent of the system in each village or the number of services in use in each.

Findlay.—The public water-supply system of Findlay (population, 17,613) was installed by the city in 1889 and has cost to date $440,250.

The supply is derived from a basin located at the side of Blanchard River and filled from that stream by seepage through a wall of filtering material. Just below the basin a dam has been built across the river in order to keep up its level in times of low water. This dam is constructed of stone and is 200 feet long, 8 feet 3 inches high, and backs the water up about 3 miles. The filtering material consists of 10 feet of gravel and charcoal placed in openings between the arches of a masonry bridge or dam built across the end of the reservoir. This wall is 295 feet long, and the supports for the arches take up 116½ feet, leaving 178½ feet for the length of the filter proper. As the filter is 6 feet deep its area is 1,071 square feet. The gravel is kept in place by screens extending across the openings of the arches, and a mass of charcoal forming a V-shaped wedge is placed between the banks of gravel. Thus but a small amount of charcoal lies at the bottom of the filter and the resistance to the flow of water is therefore less here, so that most of the water passes through gravel only and receives but little purification. The filter is cleaned three times a year by pumping water through it. The rate of filtration is controlled by gates that can be raised or lowered over the openings. The filter was designed to free the water from mud, and it does this to some extent, but the filtering media become so thoroughly impregnated with filth that, in the absence of any adequate method of cleaning, the whole operation must be looked upon with distrust. The water in the basin never appears to be muddy, but during low water a scum gathers on its surface, and it has a disagreeable odor. The reservoir is cleaned biennially by draining out the water and flushing and scraping out the mud.

The present source of supply is very much polluted by waste from the oil fields and glycerine factories that abound along the banks of the river. At present a new conduit is being built to a ridge 9 miles east of town, where an abundance of good water is found.

The water is pumped from the reservoir directly to the mains by two horizontal compound duplex Gordon pumps, each of 2,500,000 gallons capacity. There are 29 miles of mains laid, which supply 80 per cent of the population with water. There are 2,137 service taps installed, of which 1,262 are in use, and 314 of these are metered. The average daily consumption is 900,000 gallons, or 45 gallons per capita of the total population, and 140 gallons for each consumer. During the summer, when the reservoir is low, the public water supply is not generally used for domestic purposes, resort being then had to private wells.

Marietta.—The public water-supply system of Marietta (population, 13,348) was established by the city in 1890, and has cost to date $267,145.67. The supply is taken directly from Ohio River, above the town, through three intake pipes, each 275 feet long, and 8, 16, and 18 inches in diameter, respectively. When the plant was first put in it was intended to obtain a ground water from a large well near the river, but the quantity thus obtained was too small and was also objectionable owing to an excess of iron. The lower portion of this large well was filled and in the remaining portion was set a horizontal pump, so as to bring it within reach of the low stages of the river.

The nearest serious sewage pollution of Ohio River above the intake is received from the cities of Bellaire, Wheeling, Martins Ferry, and Bridgeport, about 75 miles above. A number of villages below these send some sewage to the river, but the amount is small.

The water is pumped from the river to two tanks by a 750,000-gallon horizontal duplex Gordon pump, a 1,000,000-gallon vertical compound duplex Gordon pump, and a new 3,000,000-gallon vertical compound duplex Snider-Hughes pump. The tanks are located on a hill back of the city at such an elevation as to give a pressure of 85 pounds at the pumping station. They consist of two open steel tanks, each 66 feet in diameter and 30 feet high, having a combined capacity of 1,500,000 gallons. There have been 28½ miles of mains laid, to which 95 per cent of the total population have access; 2,340 service taps have been connected, of which 2,250 are in use, and 75 of these are metered. The great quantity of sand suspended in the water cuts out the meters in a short time and they become inaccurate.

The average daily consumption is estimated at 2,700,000 gallons, or 202 gallons per capita of the total population, and 213 gallons per capita of the population actually supplied. The public water supply is in general use, but there is a great deal of complaint on account of its muddy condition during the greater part of the year. Very little relief in this respect can be expected with the present reservoir, owing

to its small storage capacity, for it requires a long time to enable the mud to settle. A new reservoir, with a capacity of 20,000,000 gallons, is being constructed, which will afford about 125 pounds pressure.

Napoleon.—The public water-supply system of Napoleon (population, 3,639) was installed by the village in 1895, which, with an electric-light plant, put in at the same time, cost $59,589. The cost to date has amounted to $103,000.

The supply is taken from Maumee River at a point opposite the upper end of the town and the backwater of the Grand Rapids dam. The water is in danger of contamination from sewage emptying just below the intake, especially when the wind is upstream, and it is always polluted by the sewage of Defiance, only 18 miles above.

The water is pumped to a standpipe by two vertical compound Smith-Vaile pumps of 1,000,000 gallons capacity each. The standpipe is 125 feet high and 20 feet in diameter, and has a capacity of 290,000 gallons. The mains are 8 miles long and supply 85 per cent of the population with water. There are 494 service taps connected, 454 of which are in use, and 111 of which are metered. The average daily consumption is 282,191 gallons, or 77 gallons per capita of the total population, and 91 gallons for each consumer. The water of the public supply is not used to any extent for domestic purposes.

Piqua.—The waterworks at Piqua (population, 12,172) were put in by the city in 1876. The waterworks proper have cost the city to date $131,838.59, and the hydraulic canal $269,000.

The supply is from the "hydraulic," a branch and feeder of the Miami and Erie Canal. This branch leaves the main canal at Lockington and joins it again below Piqua, gathering up much surface drainage on the way and furnishing a supply of water to Piqua; also the power to pump it and to run several manufacturing plants. Just above the waterworks the canal broadens into three reservoirs, in all about 100 acres in area. The reservoirs and the canal are badly overgrown with weeds and moss, as the sides and bottom are not paved in any manner. The water is pumped from the branch directly to the mains by two power pumps. One of these is a Flanders pump of 2,000,000 gallons capacity and the other a Gordon-Maxwell pump of 1,500,000 gallons capacity.

Though the waterworks trustees have developed a ground supply which secured the approval of the Ohio State board of health, owing to local jealousies and difficulties this water has not been substituted for that of the "hydraulic."

The trustees put down twenty-three 4-inch wells in the bed of Great Miami River north of the city and in the valley of Ross Creek adjoining. These wells were from 48 to 134 feet deep, driven through various strata of soil, clay, gravel, and sand. A good flow of water was found in fourteen of the wells, and these were reserved for the public supply. While there are certain local conditions which might inter-

fere with the continued purity of the water, they are so unimportant and so easily overcome that the well supply should be utilized at once, for it is vastly superior to the water from the "hydraulic."

The secretary of the waterworks, Mr. W. B. Mitchell, in his last report gives figures on the original cost of the plant and "hydraulic" and the cost of additions:

Original cost of "hydraulic"	$268,911.64
Original cost of waterworks	56,896.84
Total	325,808.48
Additional cost of waterworks	63,896.84
Total cost of plant complete to May 1, 1900	389,705.32

In addition to the above the new wells cost $2,861.71, all of which came from the revenue of the plant. The total cost of operating the plant, maintenance of "hydraulic," etc., exclusive of interest, was $5,144.32 for the last year.

There have been 24.5 miles of mains laid, giving about 90 per cent of the people access to the water. There are 1,400 services connected, of which 1,320 are in use and none metered. The average daily consumption is estimated at 1,500,000 gallons, which is 1,136 gallons per service, 227 per consumer, and 123 per capita of the total population. This large consumption is due to the general industrial use of the water and to excessive wastage, brought on by the absence of meters and the poor quality of the water.

The city water is not used for domestic purposes to any extent, the main dependence being placed on private wells. About two-thirds of these are shallow dug ones, from 15 to 30 feet deep, getting their water in a bed of gravel. The others are drilled and go 10 to 12 feet into the limestone, which is about 25 feet below the surface.

Toledo.—The public water-supply system of Toledo (population, 131,822) was put in by the city in 1872, and it is estimated that the plant has cost to date $1,979,408.78.

The supply is now, and always has been, taken from Maumee River at a point opposite the upper end of the city. Owing to the objectionable character of this water, from mud and sewage, it has been proposed a number of times to obtain a new supply or filter the present one. One of the most feasible schemes presented is to collect the underflow of the river in a number of galleries or wells placed in a large sandy bar or island opposite the present works.

At the present time the water is led directly from the river through a 5-foot steel conduit to a pump well, and from there is raised to a regulating standpipe by four Worthington and one Knowles pumps, having in all a total pumping capacity of 45,000,000 gallons. Two horizontal compound duplex Worthington pumps of 5,000,000 gallons capacity were put in in 1873. The Knowles is a horizontal compound duplex pump of 5,000,000 gallons capacity, and was installed in 1886.

The other Worthington pumps are vertical compound duplex high-duty pumps of 15,000,000 gallons capacity each, one of which was erected in 1893 and the other in 1898. The standpipe consists of a steel tank inclosed in a brick tower. The former is 225 feet high, 5 feet in diameter, and has a capacity of 42,000 gallons.

About 183 miles of mains have been laid, which give 75 per cent of the total population access to the water. There are 13,961 service taps in use, of which 7,200 are metered. The average daily consumption is 9,074,900 gallons, or 70 gallons per capita of the total population, and 92 gallons per capita of the population actually supplied.

The public water supply is not used so extensively as it should be in a city of this size, and it probably will not be until water of a better quality is supplied. Many use the small house filters, but these afford but little relief from the objectionable features of the water.

Upper Sandusky.—The public water-supply system of Upper Sandusky (population, 3,355) was put in by a private company in 1889, and to July, 1898, had cost about $70,000.

The supply is taken directly from Sandusky River at a point just above the village. When it reaches here the river has received the combined sewage of Bucyrus and Crestline, 24 and 40 miles above, respectively. This long flow has caused all visible traces of sewage to disappear, but it has by no means completely purified the water.

At the waterworks the river is intercepted by a small dam, which forms an impounding reservoir having a capacity of about 500,000 gallons. From this backwater the intake pipe leads to a small sand filter that is connected by a bypass directly with a pump well, which furnishes by far the most of the water used. It is claimed that the direct connection to the river is used only when the stream is at a good stage, and that when the water is muddy it is pumped to a reservoir, allowed to stand a certain length of time, then passed to the filter, and then to the pump well.

The storage reservoir was built in 1895, and is 325 feet long, 180 feet wide, and will hold 2,288,000 gallons. It is formed by throwing up earthen embankments around a piece of low ground and is not lined or paved in any way. All the surface drainage, except that from the immediate vicinity of the basin, is diverted by a ditch dug along the hill above.

The filter box is 11 by 20 feet and contains 3 feet of fine gravel. It is cleaned monthly by scraping off the top layer, and twice a year by washing all the gravel. It acts as a strainer only, and can improve the water but little, except as regards appearance.

The pump well, which is walled up with rough stone, is 85 feet in diameter and 12 feet deep. At first this well supplied the village with ground water, but the supply soon fell short and it was necessary to use river water. Now the river water is used almost entirely, this well supplying a small proportion only.

The water is pumped to an elevated tank by two horizontal duplex Blake pumps, each of 1,500,000 gallons capacity. This tank is constructed of wood, and is 24 feet high and 28 feet in diameter, with a capacity of 95,000 gallons. It is supported on a brick tower 75 feet high.

There have been 11 miles of mains laid, which supply practically the whole village with water. There are 500 service taps connected, of which only 350 are in use, and 30 of these are metered. The average daily consumption is about 350,000 gallons, amounting to 70 gallons per capita, or 104 gallons per consumer. The water is not used to any extent for domestic purposes, wells and springs supplying the drinking water.

Zanesville.—A public water supply was put in by Zanesville (population, 23,538) in 1840, and has cost to date $800,000. The first supply was taken direct from Muskingum River at a point which is now within the limits of the city. As most of the city's growth was upstream, the water entering the old intake became contaminated, and in 1873 the pumping station was moved to the present site above the city.

The water is pumped directly from the river to a reservoir and standpipe, and a second reservoir is filled from the first by gravity, thus giving three levels, which are necessary on account of the extreme range in the level of the city. The first reservoir built is now abandoned; the second, built by the side of it on Harvey Hill, is still in use, supplying the lowest service in the city. It has a capacity of 2,000,000 gallons, and is filled by gravity from the new reservoir on Blandy Hill, which supplies the middle service and has a capacity of 2,500,000 gallons. The high service is supplied by a standpipe built in 1894 on the same hill. The standpipe is 100 feet high, 25 feet in diameter, and has a capacity of 367,000 gallons. It is now filled directly from the river, but was formerly supplied from the new reservoir by a 1,000,000-gallon Knowles pump.

The reservoirs are completely lined with brick laid in cement, and are carefully protected for surface drainage. Owing to the large amount of sediment carried by Muskingum River, the reservoirs have to be cleaned at least twice a year.

The pumping station contains three pumps—an old-style single-acting Worthington, installed in 1873, but since remodeled and compounded, now giving a daily capacity of 3,500,000 gallons; a horizontal compound duplex Holly pump of 5,000,000 gallons capacity, put in in 1873; and a horizontal compound duplex Gordon pump of 4,000,000 gallons capacity, installed in 1895.

There are 66.5 miles of mains, which supply 90 per cent of the population with water. There are 6,500 service taps recorded in the books of the water department, the high number being due to a peculiar method of numbering, some families having several services. Of this

number 125 are metered. The average daily consumption is 4,500,000 gallons, or 191 gallons per capita of the population of the city and 216 gallons for each consumer. The water is in general use for domestic purposes, except in times of high water, when the large amount of mud makes it undesirable. The use of a large amount of water for industrial purposes accounts for the extraordinary consumption.

GROUND SUPPLIES.

SUBSURFACE.

Ashland.—The public water-supply system of Ashland (population, 4,087) was installed by the village in 1895, and has cost to date $70,000. The supply is derived from two wells situated in low land lying between the branches of Jerome Fork, about one-half mile east of the corporation limits. One of the wells is 30 feet in diameter and 16 feet deep; the other is 36 feet in diameter and 18 feet deep. Both are constructed of brick, roofed over, and fully protected from surface drainage. The smaller well was dug in 1895, when the works were first put in, but it soon failed to furnish enough water, and the larger well was dug the next year.

The water is obtained from a bed of gravel which is found just below the surface soil. During wet seasons the water rises to the surface, but in dry seasons it falls to within a few feet of the bottom of the wells. The water is pumped from the wells through the village to a standpipe beyond by two horizontal compound duplex Smith-Vaile pumps of 1,000,000 gallons capacity each.

The standpipe is 20 feet in diameter and 75 feet high, and has a capacity of 176,000 gallons. The average pressure on the street main is 75 pounds and at the station 122 pounds.

There have been 15 miles of mains laid, giving 80 per cent of the inhabitants access to the water.

There are 600 service taps connected, of which 500 are in use, and 250 of these are metered. The average daily consumption is 204,350 gallons, or 50 gallons per capita of the population of the village, and 62½ gallons per capita of the persons actually supplied.

Attica.—The public water-supply system of Attica (population, 694) was installed by the village in 1895, and has cost to date $17,147. The supply is from a well 28 feet in diameter and 25 feet deep, fed by an 8-inch tile collecting gallery 300 feet long and laid in an extensive bed of gravel. The well is lined with brick and covered with a board roof. The well and gallery are removed from local pollution and seem to furnish an excellent and abundant water supply.

The water is pumped to an elevated tank by a Gordon pump of 500,000 gallons capacity, operated by a Hicks gasoline engine. The tank, which stands on a brick tower 40 feet high, is 20 feet in diameter and 25 feet high, and has a capacity of 59,000 gallons.

There are 2½ miles of mains laid, which supply water to about 95

per cent of the population. There are 100 service taps connected, all of which are in use, and 67 of these are metered. The average daily consumption is 10,000 gallons; the average consumption per capita of the village is 14 gallons, and the average per consumer is 15 gallons.

Barberton.—The public water-supply system of Barberton (population, 4,354) was put in by the village in 1892, and has cost to date $45,000. The cost represents mains and hydrants only, as the water is purchased by the city from three large manufacturing establishments, which not only furnish the water but also supply the pressure. These plants have one well each, all of which are about 14 feet deep and 16 feet in diameter. They are located in beds of gravel near the canal.

The water is pumped directly to the mains by four pumps located in the various factories, one of which maintains the pressure, the others being held for use in emergency. The pumps consist of a horizontal compound Worthington of 640,000 gallons capacity, a Hooker pump of 320,000 gallons capacity, and two others of 512,000 gallons each, making a total daily capacity of 1,984,000 gallons.

There have been 9 miles of mains laid, to which 60 per cent of the people have access. There are 425 services connected, all of which are in use and none metered. It is impossible to estimate the average daily consumption, but it is not important, as this supply is not used at all for domestic purposes. A potable water supply is projected, and it is highly desirable that such a supply be obtained.

Bellefontaine.—A system of public water supply was put in by Bellefontaine (population, 6,649) in 1882, and has cost to date $140,000. The water is obtained from four 8-inch wells located just south of the village, in the valley of a small stream. The wells are 160 feet deep and go through 80 feet of gravel and clay and 80 feet into limestone. The supply has always been sufficient, but it is intended to put down a number of shallow wells in the gravel for an additional supply. There is no local pollution of the ground, and even if there were it is a question whether it would affect the deep wells.

The water is pumped direct from the wells to a reservoir by two horizontal compound duplex Gordon pumps of 750,000 gallons capacity each. A new vertical compound duplex Snow pump of 2,500,000 gallons capacity is to be installed soon for reserve in case of accident to the present pumps. The reservoir is located on high ground to the east of the village, and gives a pressure of 80 pounds at the station and an average of 60 pounds over town. It is an open basin, 206 by 106 by 20 feet, holding 2,000,000 gallons, with sloping sides built of brick laid in cement. The whole is surrounded by a high barbed-wire fence for protection from contamination.

There are 17 miles of mains in, to which about 95 per cent of the people have access. There are 956 services, of which 900 are in use and only 18 are metered. It is intended to increase the number of

meters very soon, so as to cut down waste, as the consumption is very large for a village of this size. The average daily consumption is 642,000 gallons, which is 714 gallons per service, 143 per consumer, and 94 per capita of the total population.

Canal Dover.—A public water-supply system was installed by Canal Dover (population, 5,422) in 1895, and has cost to date $40,152.10. The water is obtained from six 6-inch driven wells, ranging from 51 to 61 feet deep, penetrating a large bed of sand and gravel, located in the southern part of the village on the bank of Tuscarawas River. Four of the wells were dug when the plant was installed, in 1895, and two more were added in 1897. The water rises to within 10 feet of the surface when the wells are not being pumped, and it is lowered but little by pumping.

The water is pumped to a standpipe by two horizontal compound duplex Worthington pumps of 750,000 gallons capacity each. The standpipe is 100 feet high and 16 feet in diameter, giving a capacity of 150,000 gallons and a pressure, when full, of 60 pounds per square inch at the station.

There are 10 miles of mains laid, to which 87½ per cent of the people have access. There are 742 service taps connected, of which 700 are in use and 8 are metered. The average daily consumption per capita of the village is 27 gallons and per consumer 31 gallons. This supply is of good quality, but is not in general use.

Columbus Grove.—A public waterworks has been installed at Columbus Grove (population, 1,935). The supply is obtained from wells drilled into the limestone found a short distance below the surface. The water is pumped from the wells to a reservoir of 60,000 gallons capacity by a McGowan compound duplex pump of 75,000 gallons capacity. About 4¾ miles of mains have been laid, to which 85 per cent of the population have access. Thirty service taps have been connected, all of which are in use and metered. As the plant has just been installed, it is impossible at this time to estimate the daily consumption.

Coshocton.—A public water-supply system was put in by Coshocton (population, 6,473) in 1892, and has cost to date $100,000. The supply is from twenty-two 6-inch wells, driven from 31 to 32 feet into the sandy point between Walhonding Creek and Tuscarawas River. The level of the water in the wells is the same as that in the streams, usually about 8 feet below the surface, and it is lowered but little during pumping. The water is pumped directly from the wells to a covered brick reservoir of 324,000 gallons capacity, which is located on a hill east of the town, of such height as to give a pressure of 115 pounds. The pumps in use are a vertical compound duplex Hughes of 1,250,000 gallons capacity and a horizontal compound duplex Hughes of 600,000 gallons capacity.

There are 16 miles of mains laid, giving 90 per cent of the people access to the water. There are 1,000 service taps connected, all of

which are in use, and 25 are metered. The average daily consumption is 1,500,000 gallons, or 232 gallons per capita of the village and 257 gallons per consumer.

Dayton.—In 1869 a public water-supply system was installed by Dayton (population, 85,333), which has cost to date $1,811,618.47 for construction alone. The total expenditure of the waterworks department for thirty years was $2,108,721, of which $745,429 was for maintenance. The total income for the same period was $1,248,638, showing the net cost of the plant to be $860,083, a figure much below the present value of the works. The construction account for 1900 was heavy, as a new suction was put down, also new wells, and a large pump is to be put in, which will require extensive changes in the pumping station.

The first supply was directly from Mad River, but this was abandoned in 1887, when the first of the present extensive system of wells were put down. There are now eighty-seven 8-inch wells, driven 30 to 60 feet in the bed of Mad River in the upper portion of the city. During the last year twenty-four of the old wells were replaced by twenty additional new ones, bringing the total to the figure named above.

The bed of Mad River consists of a porous gravel which carries a large amount of water, the subsurface flow of the stream. The normal level of the water in the wells coincides with the river level except when the stream is at very low stages, when it rises above the surface water level. On a test one well gave 400 gallons per minute, the level being lowered only 5 feet. At this same test a well 50 feet away was lowered only 4 inches. The wells extend up and down the river 2,000 feet each way from the pumping station, and until this year were reached by a 30-inch suction line going down the river and by a 20-inch and 30-inch going up. A new 5-foot suction line is being put in up the river to replace the small lines, and also, in time, to extend above them as new wells are added.

While the site of the wells is above all sewage pollution, the water does not escape all manufacturing refuse, especially that from a strawboard works. Except in times of extreme low water it is doubtful whether any of this refuse would influence the subsurface water, but at such dry times some of it would soak into the gravel bed, where, if the filtration were not perfect, it would pollute the water of the wells.

The water is pumped directly to the mains by a 4,000,000-gallon quadruplex Holly pump, installed in 1873; a 10,000,000-gallon horizontal compound duplex Holly pump, erected in 1899, and a 15,000,000-gallon pump of the same pattern, installed in 1895. A new 10,000,000-gallon vertical triple-expansion Holly is now being put in.

The domestic pressure averages 60 pounds. About 130 miles of mains have been laid, giving about 90 per cent of the total population access to the water. There are 14,800 service taps connected, and 12,500 in use, and 8,800 of these are metered, about 60 of which are

for hydraulic elevators. The average daily consumption is 7,000,000 gallons, which is 91 gallons per consumer, and 82 gallons per capita of the total population. The water is of excellent quality, and has come into general use, to the gradual exclusion of local wells.

Eaton.—Waterworks were put in by Eaton (population, 3,155) in 1891, and have cost to January, 1903, $91,200.47. The supply is from wells and springs near Sevenmile Creek. The well water is furnished by ten driven wells, ranging in depth from 74 to 89 feet and all 6 inches in diameter. They go through 55 feet of hardpan into a bed of gravel, which furnishes the water. The wells flow when not pumped, and the water can be lowered 35 feet during dry weather. The springs are one-fourth mile west of the waterworks, at the foot of a hill just across Sevenmile Creek. The water is collected by a sewer-pipe gallery and is brought by gravity through a tile pipe to the station. Near the works this pipe line empties into a silt well, 13 feet in diameter and 15 feet deep, which frees the water from the heavier sediment before it reaches the pumps. It is estimated that the wells can supply 150,000 gallons per twenty-four hours and the springs 200,000 gallons. A covered reservoir of 100,000 gallons capacity is just being built, into which the water of the wells and springs may flow when the pumps are not running. The reservoir is 110 feet long, 12 feet wide, 12 feet deep, and will be built of brick laid in cement. Direct connection can be made to the creek for use in case of fire. The water is pumped to a standpipe by three pumps—two horizontal compound Gordons of 1,000,000 gallons capacity each and one small Hughes pump of 500,000 gallons capacity. The first pumps were put in with the works, and the latter in 1898. The standpipe consists of a tank 20 feet in diameter and 80 feet high, placed on top of a stone tower 50 feet high. It has a capacity of 185,000 gallons.

Franklin.—The public water-supply system for Franklin (population, 2,724) was put in by the village in 1887, and has cost to date $57,000. The supply is obtained from six 6-inch wells driven 65 feet into a clay, sand, and gravel bed, bordering the Miami and Erie Canal in the lower part of the village. The water stands normally 22 feet below the surface and is lowered but little by ordinary consumption. There is direct connection with the canal for use in cases of emergency.

The water is pumped directly from the wells to the mains by two horizontal compound duplex Holly pumps of 750,000 gallons capacity each. The pumps are set 18 feet below the surface, so as to keep the water well within the suction limit. The station pressure carried is 60 pounds and the average on the mains is 55 pounds.

There are 8 miles of mains laid, giving about 90 per cent of the people access to the water. There are 410 services connected, of which 340 are in use, and 24 of these are metered. The average daily consumption is estimated at 300,000 gallons, which is 882 gallons per service, 172 per consumer, and 110 per capita of the total population.

The large consumption is due chiefly to the extensive use of the water by the various paper mills, but in some measure to the large amount of leakage claimed to be caused by the deterioration of the pipes from electrolysis.

Granville.—A public water-supply system was installed by Granville (population, 1,425) in 1884, and has cost to date $15,000. The supply is from three 4-inch wells driven 45, 50, and 55 feet into a bed of gravel located south of the village, in the valley of Raccoon Creek. The water is pumped directly from these wells to an elevated tank by a horizontal compound duplex Knowles pump of 350,000 gallons capacity.

The tank is 32 feet in diameter, 20 feet high, and has a capacity of 100,000 gallons. It is constructed of wood and is placed on a frame trestle located on a hill just northwest of the village. The domestic pressure is 80 pounds.

There have been 6¾ miles of mains laid, giving 80 per cent of the people access to the water. There are 206 services connected, all of which are in use, and 6 are metered. The average daily consumption is 50,000 gallons, which is 28 gallons per capita of the village and 40 gallons for each consumer. The water is in general use throughout the village.

Greenville.—In 1893 waterworks were put in by Greenville (population, 5,501), and have cost to date $110,000. The original supply was derived from nine wells 6 inches in diameter and from 50 to 57 feet in depth; but lately six new 8-inch wells, averaging in depth from 46 to 54 feet, were added as a safeguard against the failure of the first set. The wells go through gravel and clay into a bed of gravel. The water rises nearly to the surface when the wells are not being pumped, and during a test it was lowered 18 feet, at which depth pumping at the rate of 900,000 gallons per day failed to lower the water farther. The waterworks are west of town, on Greenville Creek, to which direct connection can be made in case of fire, but this connection has not been used since August, 1895. There is a milldam on Greenville Creek, 1½ miles above the waterworks, which controls the amount of water in the creek during dry weather. It is 6 feet high, 40 feet long, and backs the water up about 1½ miles.

The water is pumped directly from the wells to a standpipe by two horizontal compound duplex condensing Hughes pumps of 1,500,000 gallons capacity. The standpipe consists of a tank 60 feet high and 20 feet in diameter, placed on a brick tower 70 feet high. It has a capacity of 141,000 gallons.

Thirteen miles of mains have been laid, which supply 80 per cent of the population with water. There have been 961 services connected, 811 of which are in use, and 540 of these are metered. The average daily consumption is 400,000 gallons, which is 91 per consumer and 73 per capita of the whole population.

Hamilton.—A public water-supply system was installed in 1884 by Hamilton (population, 23,919), and has cost to date $365,000. The first supply was derived from a large well or basin situated in the low land north of the city, lying between the Great Miami River and an abandoned channel of the same stream. The basin was 500 feet long, 100 feet wide, and 15 feet deep, dug into a large bed of gravel.

The supply from this source was unsatisfactory, and in 1889 a number of deep wells were driven in the basin, and these, with additions, have furnished the supply since that time. Now there are nineteen 6-inch and three 8-inch wells, driven from 75 to 135 feet into the bed of gravel.

The water flows from the wells at the river level and is conducted about 1,000 feet to a receiving basin set 13 feet below mean low water in the river. The receiving well is a closed brick chamber 28 by 13 by 33 feet deep, with a capacity of 90,000 gallons.

The water is pumped from this basin to a supply reservoir by a vertical compound duplex Gordon-Maxwell pump of 3,000,000 gallons capacity. The average domestic pressure is 90 pounds. The reservoir is situated on the high ground west of Great Miami River, and is reached by two mains, a 20-inch main laid in the bed of the river and a 10-inch main crossing on a bridge. It is an open basin, with sides paved with brick in cement, and is 200 by 270 by 24 feet deep, with a capacity of 7,000,000 gallons. As first constructed the reservoir was not cemented and there was trouble frequently with vegetable growths of various kinds. Since the sides were cemented, however, no trouble has been reported.

There are 40 miles of mains laid, giving about 95 per cent of the population access to the water. The mains even extend outside of the corporation in order to supply several manufacturing plants. There are 3,295 services in use, and 827 of these are metered. The average daily consumption is 1,818,000 gallons, which is 569 gallons per service, 103 per consumer, and 76 per capita of the total population. It will be noted here that the pumping capacity is less than twice the average daily consumption, a rather perilous condition for a pumping plant, notwithstanding the very fair reservoir capacity.

Lynchburg.—A public water-supply system was installed in 1896 by Lynchburg (population, 907), and had cost to January, 1901, $18,000. The first supply was obtained from a dug well 16 feet in diameter and 20 feet deep, which enters a bed of gravel just above the rock. This well soon failed to supply enough water, and a 4-inch suction line was run to a large basin from which a distillery obtains its water supply.

This basin consists of a trench 120 feet long by 15 feet wide, with an 18-foot well at each end, all excavated 15 feet deep in a bed of sand and gravel. The sides of this basin are planked, and the whole is covered with a flat plank roof at the ground level. It occupies a low swale and is easily accessible to surface drainage, as the well is situ-

ated nearly in the center of the village. In 1900 the 4-inch suction line to the distillery basin was replaced by a 5-inch line.

As the distillery required the entire yield of this basin, the village was compelled to seek a new supply. Application was made to the State board of health for the approval of three springs, only one of which could be approved, namely, the "sulphur" spring, located 1¼ miles northeast of the village, in low land near a small run. With proper care this spring ought to be able to provide at least 20,000 gallons of potable water a day. It is at such a height that the water could be conducted by gravity to a pump well at the present station.

The water is pumped from the well and basin to an elevated tank by a horizontal duplex Laidlaw-Dunn-Gordon pump of 250,000 gallons capacity. The tank is 16 feet in diameter, 20 feet high, with a capacity of 30,000 gallons, and is placed on a trestle 89 feet high. The average pressure on the mains is 40 pounds.

There are 4½ miles of mains in use, to which all the people have access. There are 135 services in, of which 122 are in use, and none is metered. The average daily consumption is estimated at 25,000 gallons, which is 53 gallons per consumer. The water is not in general use for domestic purposes.

McConnelsville.—A public water-supply system was installed in 1899 by McConnelsville (population, 1,825), and has cost to date $30,000. The supply is from four 6-inch wells, 42 feet deep, located in the river bottom just above the village. The wells go through 18 feet of loam, sand, and gravel and a thin layer of hardpan, then into the water-bearing gravel. The normal level of the water in the wells is 17 feet below the surface, and it is lowered but little by pumping. The water is pumped directly from the wells to an open brick reservoir by horizontal duplex Stillwell-Bierce Smith-Vaile pumps of 750,000 gallons capacity. The reservoir is placed on a hill back of the village, where it gives a pressure of 100 pounds.

There are 3¾ miles of mains laid, to which 80 per cent of the people have access. There are 200 services connected, of which 175 are in use, and none is metered.

Middletown.—A public water-supply system was installed in 1874 by Middletown (population, 9,215), and has cost to date $450,000. The supply is obtained from a large well 20 feet in diameter and 35 feet deep, dug in a gravel bed in the upper part of the city, near Great Miami River. The normal level of the water is 16 feet below the surface, and it is lowered about 11 feet by ordinary consumption in the dry season. The supply has always been sufficient except in 1893, when it failed during a period of exceptionally dry weather.

The water is pumped directly from the well to the mains by a quadruplex Holly power pump, installed in 1874. The pump has a capacity of 1,250,000 gallons per day and is operated by a 56-inch American

turbine, operating under a 17-foot head of water, secured from the Middletown ditch or "hydraulic."

In 1885 two horizontal compound duplex Smith-Vaile steam pumps of 1,500,000 gallons capacity each were installed. These are for reserve and for use in case of fire, steam being kept at low pressure in one boiler all the time. In cases of emergency the water from the "hydraulic" can be admitted to the well.

Enough water power to supply the domestic pressure is obtained from the "hydraulic" at an annual rental of $260, certainly a very economical method of pumping. A pressure of 45 pounds is maintained at the station, and the average on the mains is 40 pounds.

There have been 20 miles of mains laid, to which about 87 per cent of the people have access. There are 1,337 services in, of which 1,300 are in use, and 62 of these are metered. The average daily consumption is 1,500,000 gallons, 187.5 per consumer, and 163 per capita of the total population. The large consumption is due to some extent to the general use of the water for industrial purposes, also to an extensive waste, induced by cheapness of the water.

Millersburg.—A public water-supply system was installed in 1895 by Millersburg (population, 1,998), and has cost to date $27,000. The water is obtained from a well 30 feet in diameter and 28 feet deep, located just west of the village, in bottom land, 50 feet from Killbuck Creek. The well is walled up with brick and is covered with a conical board roof. It goes through loam and muck into a bed of gravel, where the water is found. The normal level of the water is above that in the creek, varying from 15 to 24 feet from the surface, and is seldom lowered below the latter figure. The water is pumped directly from the well to a covered reservoir by two horizontal compound duplex Stillwell-Bierce pumps, each of 750,000 gallons capacity. The reservoir is constructed of brick and covered with a tin roof, and has a capacity of 150,000 gallons. It is located on a hill just east of the village, and gives an average pressure of 80 pounds.

There are 6 miles of mains laid, to which 95 per cent of the people have access. There are 400 services connected, all of which are in use, and 5 are metered. The average daily consumption is 500,000 gallons, which is 250 gallons per capita of the total population and 263 gallons for each consumer.

Mineral Point.—A public water-supply system was installed in 1899 by Mineral Point (population, 1,200), and has cost to date $12,000. The water is obtained from four springs, is impounded in a reservoir of 169,000 gallons capacity, and is supplied to the village by gravity at a pressure of 80 pounds. The reservoir is 60 feet in diameter, 8 feet deep, and is built of brick and covered with a slate roof. The springs are located under steep hills rising in a thinly settled farming district about 1 mile northeast of the village. For fire purposes a reserve supply is stored in an open reservoir of 88,000 gallons

capacity, which is located in the village. The pressure is obtained by a horizontal duplex Battle Creek pump of 720,000 gallons capacity.

There are 3 miles of mains installed, giving 90 per cent of the people access to the water. There are 100 service taps, all of which are in use, and none is metered. It is impossible to estimate the amount of water used.

Minerva.—A public water-supply system was installed in 1886 by Minerva (population, 1,200), and has cost to date $16,000. The supply was first obtained entirely from Clear Fork of Sandy Creek; then from an infiltration well placed at the side of this creek in the upper part of the village; then this supply was supplemented by driving a 6-inch well 55 feet deep into sand and gravel. The infiltration well is 9 feet square, boarded up with plank, and 10 feet deep. The water enters it through quicksand at the bottom. At times the creek covers this well and the deep well is used entirely. Normally each well supplies about half of the water. When not pumped, the water stands 6 feet from the surface, or a little above the creek level, and by pumping is easily lowered to the suction limit. The water is pumped directly from these wells to an elevated tank by a horizontal duplex Smith-Vaile pump of 250,000 gallons capacity and by a horizontal compound duplex Worthington pump of 600,000 gallons capacity. The tank is 20 feet high and 24 feet in diameter, giving a capacity of 65,000 gallons, and is placed on a frame trestle located on a hill north of the village, where it gives a pressure of 50 pounds.

There are 5½ miles of mains laid, giving 90 per cent of the population access to the water. There are 250 services connected, all of which are in use, and none is metered. The average daily consumption is 201,000 gallons, which is 167½ gallons per capita of the village and 170 for each consumer. The public supply is in general use and seems to be of good quality.

Mount Vernon.—A public water-supply system was installed in 1882 by Mount Vernon (population, 6,633), and has cost to date $115,000. The original supply was from two large wells—one 20 feet in diameter and the other 7 feet, and both 21 feet deep. The larger of these wells has been filled up until it is now only 10 feet deep and is used as a receiving well. The smaller is used as a pump well, the water flowing from the receiving well to it by gravity.

The supply is now obtained from 21 driven wells, which have been put in from time to time as the consumption increased. These consist of nine 6-inch, four 4-inch, eleven 2-inch, and two 3-inch wells, which are from 60 to 90 feet in depth, the average being 73 feet. The wells go through a surface layer of sand and gravel, then through a layer of hardpan and into water-bearing gravel. The wells were all artesian and the natural flow only is utilized; five, however, have now ceased to flow. The sand found through the gravel stops the flow in a comparatively short time and the wells have to be blown out

to keep the flow constant. The supply is now just about equal to the demand, and it is intended to sink more wells.

The water is pumped to a standpipe by a horizontal compound duplex Worthington pump of 750,000 gallons capacity and a horizontal compound duplex McGowan pump of 1,000,000 gallons capacity. The standpipe is 25 feet in diameter and 57 feet high, has a capacity of 209,290 gallons, and furnishes an average pressure of 100 pounds.

There are 20 miles of mains laid, giving 80 per cent of the people access to the water. There are 1,100 services connected, of which 1,025 are in use, and none is metered. The average daily consumption is 1,000,000 gallons, which is 151 gallons per capita of the village and 188 for each consumer. The wells are located in the western part of the village, in the bottom land of the Kokosing River, a large area of which is under the control of the village and is laid out for a public park, so as to protect the water supply.

New Philadelphia.—The public water-supply system of New Philadelphia (population, 6,213) was put in by a private company, the New Philadelphia Water Company, in 1886, and has cost to date about $100,000.

The supply was first taken from an infiltration well at the edge of Tuscarawas River, just south of the town. In 1892 eight 6-inch wells, 30 feet deep, were put down in a large gravel deposit in the lowland along the river and near the old infiltration well. In 1896 six 6-inch wells, 80 feet deep, were put down in the same vicinity, and in 1898 two more of the same kind were put in. The normal level of the water is 11 feet below the surface, and it is lowered 15 to 18 feet by pumping.

The old infiltration well is now connected directly with the river, and can be drawn upon in case of fire. The water is pumped directly from the driven wells to a reservoir by two horizontal duplex Blake pumps of 1,000,000 gallons capacity each and by one horizontal compound duplex Laidlaw-Dunn-Gordon pump of 2,000,000 gallons capacity. The reservoir is located on a hill east of the village, at such a height as to give a pressure of 100 pounds. It is oval in shape, 200 by 100 feet, is 13 feet deep, and has a capacity of about 1,250,000 gallons. The bottom of the reservoir is lined with a mixture of cement and clay, the sides are riprapped, and the whole is covered with a board roof.

There have been 15 miles of mains laid, to which 90 per cent of the people have access. There are 600 service taps, 575 of which are in use, and 8 are metered. The average daily consumption is 600,000 gallons, which is 96 gallons per capita of the village and 175 gallons per consumer. The wells are removed from any local pollution and furnish good water.

Osborn.—A public water-supply system was established in 1895 by Osborn (population, 948), and has cost to date, including the electric-

light plant, $30,000. The supply is obtained from four 6-inch wells in the northern part of the village. The wells are driven 50 feet through a thin stratum of clay and into a large bed of gravel. The normal level of the water is from 15 to 18 feet below the surface, and it is lowered but little by pumping.

The water is pumped directly from the wells to a standpipe by a horizontal compound duplex Laidlaw-Dunn-Gordon pump of 500,000 gallons capacity. The standpipe is 10 feet in diameter, 125 feet high, and has a capacity of 70,000 gallons. The station pressure is 60 pounds and the average on the mains is 50.

There are 3 miles of mains in use, to which about 80 per cent of the people have access. There are 100 services connected, of which 97 are in use, and none is metered. The average daily consumption could not be obtained.

The water seems to be of good quality, but it is not yet in general use for domestic purposes.

Oxford.—A public water-supply system was installed by Oxford (population, 2,009) in 1896, and has cost about $45,000. The supply is obtained from a large well dug in the sand and gravel bed near Four-mile Creek, some distance northeast of the village. The well is 20 feet in diameter and 35 feet deep. It is walled up with brick laid in cement and arched over so as to prevent flooding during high water, which overflows the well site. The top of the well is provided with a ventilator above high water. The normal level of the water is 9 feet from the surface, but it is frequently lowered to 24 feet by pumping.

The water is pumped directly from the well to the mains by two horizontal compound duplex Smith-Vaile and Stillwell-Bierce pumps of 750,000 gallons capacity each. The pressure on the mains varies from 45 to 65 pounds.

There are 7 miles of mains laid, which give about 95 per cent of the people access to the water. There are 234 services connected, of which 211 are in use, and 9 are metered. The daily average consumption is 30,000 gallons, which is 142 gallons per service and 15 gallons per capita of the village. The total number of consumers is hard to estimate, owing to the fact that three large institutions are supplied, but the consumption per consumer can not be far from 25 gallons.

Piedmont.—A public water-supply system was put in at Piedmont (population, 275) by a private company, the Piedmont Water Supply Company, in 1894, and has cost to date $1,450. The supply is obtained from two springs located on the neighboring hills, the water from which is impounded in a reservoir and conducted to the village by gravity. The reservoir consists of a vaulted stone cistern, with a capacity of 12,500 gallons, and is at such a height as to give a pressure of 55 pounds on the streets.

There are 2 miles of mains in, which give about 60 per cent of the village access to the water. There are 45 services, of which 40 are in use, and none is metered. Owing to the nature of the supply, it was

impossible to estimate the consumption. The springs emerge from a bed of shaly sandstone in the side of the hills, and the water appears to be of good quality.

Shelby.—The public water-supply system of Shelby (population, 4,685) was put in by a private company, the Shelby Water Company, in 1897, and has cost to date $140,000. The supply is from twelve 6-inch wells, 46 feet deep, located in lowland lying northwest of the village and bordering Black Fork. The wells go through clay and sand, gravel, and blue clay, into a stratum of gravel, in which the water is found. The normal level of the water in the wells is 5 feet from the surface, and the supply has been so abundant that the level is seldom lowered more than a foot by the pumps. The creek at times overflows the land occupied by the wells, but it is claimed the turbidity of the well water does not increase during such times, showing no intimate connection between the surface water and that obtained from the wells.

The water is pumped directly from the wells to a standpipe by two horizontal compound duplex McGowan pumps of 2,000,000 gallons capacity each. The standpipe is 16 feet in diameter and 145 feet high, giving a capacity of 218,000 gallons and an average pressure of 1,000 pounds.

There are 15 miles of mains in use, giving 95 per cent of the people access to the water. There are 710 services connected, 640 of which are in use, and 4 are metered. The average daily consumption is 750,000 gallons, or 160 gallons per capita, or 170 gallons per consumer. The water is rather highly impregnated with iron, and on this account is objectionable to many for domestic use.

Tippecanoe City.—A public water-supply system was put in by Tippecanoe City (population, 1,703) in 1897, and has cost to date, together with the electric-light plant, $28,000. The supply is obtained from driven wells located in the eastern part of the village, near the Miami and Erie Canal. There are five 8-inch wells, driven 80 feet deep into a bed of sand and gravel. Only three of the wells are in use. The normal level of the water is 10 feet below the surface, and this is lowered by pumping to a depth of from 17 to 35 feet, according to the season.

The wells are surrounded by scattered houses, and will in time suffer pollution unless steps are taken to drain off all objectionable refuse.

The water is pumped directly from the wells to the mains by two horizontal duplex Smith-Vaile pumps of 500,000 gallons capacity each, and at a pressure of 60 pounds, which is decreased to 55 on most of the mains.

There are 6 miles of mains, which supply about 95 per cent of the population with water. There are 345 services, 310 of which are in use, and 18 are metered. The average daily consumption is 75,000 gal-

lons, which is 281 gallons per service, 56 per consumer, and 44 per capita of the total population.

Trotwood.—The smallest village—or rather community, as it is not incorporated—in the State that has a public water supply is Trotwood (population, 214). The plant was put in in 1899 by a stock company, formed of nearly all the citizens, to provide primarily for fire protection only, and has cost to date $4,000. The installation of the system was occasioned by a very disastrous fire the year previous.

The water supply is procured from two 8-inch wells, driven 26 feet through clay and hardpan and into a bed of gravel. The water stands in the wells 12 feet below the surface, and the level was lowered but little by pumping at the rate of 120 gallons per minute. The water is pumped directly from the wells to a horizontal steel tank by a vertical duplex power pump run by a gasoline engine. The tank, engine, pump, and wells are all within the confines of a neat stone building situated at the edge of the village.

The pump was made by C. O. Lucas, of Greenville, Ohio, and is fitted with a device by which a certain amount of air can be pumped with the water to the tank, and there, under pressure, force the water through the mains when the pump is not running. The pump is a double-acting one, with a capacity of 175,000 gallons in twenty-four hours, working under a pressure of from 40 to 80 pounds, the limits allowed in the tank. This tank is 24 feet long, 5 feet in diameter, and holds 5,000 gallons. It is fitted with a blow-off cock for the regulation of the pressure and the amount of air above the water.

There are 3,900 feet of mains, to which about 90 per cent of the people have access. There are 38 services connected, 30 of which are in use, and none is metered.

The water is of good quality, but it is not yet in general use for domestic purposes. On account of the few services and the wide range of pressure allowed in the tank—from 40 to 80 pounds—the pump is run for a short time only each day, and the water in the tank is not so fresh as it would be with a larger consumption.

Troy.—The public water-supply system of Troy (population, 5,881) was installed in 1884, and has cost to date $125,000. The first supply was obtained from a large well, 25 feet in diameter and 31 feet deep, dug in the sand and gravel that underlies this whole section. This failed to supply the increasing consumption, and in 1891 another large well was dug, which was later filled by a flood and not reopened. As the consumption increased, five 8-inch wells were driven 20 feet in the bottom of the first large well, and the flow considerably increased for a time, but the water soon ran low and they are not of much use now. In 1898 three wells were driven near the old large wells, but they were of no value. In the same year five 8-inch wells were driven along the street south of the plant, and considerable water was found at from 38 to 51 feet below the surface. In 1899 eight 8-inch wells were

driven just east of the plant to a depth of from 38 to 54 feet. The last two sets of wells, 13 in all, furnish the bulk of the water.

The water is pumped directly from the wells by two horizontal compound duplex Laidlaw-Dunn-Gordon pumps of 1,700,000 gallons capacity each. Until March, 1898, nearly all of the water was pumped by two Smith-Vaile power pumps of 1,000,000 gallons capacity each, actuated by a 15-inch and 28-inch Victor turbine supplied with water under 16 feet 8 inch head from the "hydraulic," a ditch fed from the Miami and Erie Canal and a little local territory, which furnished satisfactory power until March, 1898, when a flood washed out the embankment, damaged the power house, and filled up one of the wells, as noted above. The average domestic pressure carried is 60 pounds.

There are 14 miles of mains in use, to which about 90 per cent of the people have access. There are 1,002 services connected, of which 994 are in use, and 78 of these are metered. The average daily consumption is 503,000 gallons, which is 506 gallons per service, 101 gallons per consumer, and 86 gallons per capita of the total population.

The waterworks is located in the northern part of the city, where the wells are subject to but little local pollution, and the water seems to be of good quality.

Urbana.—The waterworks at Urbana (population, 6,808) were put in by a private company in 1878. The supply is from one large well 20 feet in diameter and 23 feet deep, walled up with rough stone and covered with a conical board roof, and from eight 6-inch wells averaging 40 feet in depth. The large well goes through about 4 feet of soil, 2 feet of blue clay, and then through layers of gravel and sand, finding water in a layer of fine gravel 23 feet down. The small wells go through the same strata and obtain their water in an extensive bed of gravel, which continues downward for 150 feet, when bed rock is struck.

A reservoir 175 by 80 by 13 feet deep, with a capacity of 140,000 gallons, is used to store water in case of emergency. It is claimed it has not been used for a year at least. The sides and bottom of the reservoir are carefully cemented, and practically no surface drainage has access to it. The reservoir is cleaned three or four times a year to remove moss and other vegetable growths.

The water is pumped from the large well directly to the mains by a quadruplex Holly pump of 1,500,000 gallons capacity, installed in 1878, and a horizontal compound duplex Holly pump of 2,000,000 gallons capacity, put in about 1889. There are 8.4 miles of mains in, giving nearly all the city access to the water. There are 900 services in use, of which 40 are metered. The average daily consumption is about 1,000,000 gallons, and the average per consumer is 185 gallons.

The private water supply is from dug and driven wells ranging from a few feet to 60 feet in depth. These wells obtain their water from a large bed of gravel which underlies the whole city. This bed is com-

posed of alternate layers of gravel, sand, and blue clay. The clay is not continuous, but is very much broken, especially in the western part of the city. In the eastern half the layers of hardpan and clay are more extensive, making several water-bearing strata, one above each layer of hardpan. This large gravel bed supplies an abundance of water, and would furnish a water of good quality if it were protected from pollution. Urbana has no sewers whatever. The surface drainage is to Dugan Creek and its branches, through surface drains and gutters. Dugan Creek is a tributary of Mad River. Many houses have water-closets that drain to cesspools, which are put down into the gravel, so that the water will drain off and not fill up the cesspool so quickly. As soon as they become full, new ones are dug and the old ones are covered over. Vaults are also dug down into the gravel, so as to require less frequent cleaning, or in this case renewal, as very few vaults are ever cleaned. This practice has loaded the ground water with filth, and would make well water absolutely unfit for use if it were not for the filtering action of the gravel and sand. The water is at the present time a potable one.

There are now 20 miles of mains laid, giving about 85 per cent of the people access to the water. There are 1,137 services connected, of which 1,000 are in use, and 60 of these are metered. The average daily consumption is 1,170,000 gallons, which is 1,286 gallons per service, 257 gallons per consumer, and 186 gallons per capita of the total population.

Versailles.—On November 1, 1900, Versailles (population, 1,478) obtained the approval of the State board of health for a public water supply to be obtained from three 8-inch wells located on the banks of Swamp Creek, near the center of the village, though not in the immediate neighborhood of many houses. The wells are driven 30 feet deep through a bed of clay and into gravel, and could furnish an excellent water but for contamination from the surrounding population. The water is not of the best, but as it is all that can be had, and is so much better than the private well water, its use was permitted. The plant has just been put in, and has cost to date $38,000. Three and one-half miles of mains have been laid, 90 per cent of the population having access to them, and 126 services have been connected, all of which are in use, and none is metered. The average daily consumption is 200,000 gallons, which is 135 gallons per capita of the total population, or 150 gallons per consumer.

Wadsworth.—A public water-supply system was put in at Wadsworth (population, 1,764) by a private company in 1881, and has cost to date about $12,000. The supply was first obtained by gravity from three springs, two situated on the high ground just north of the village and one on a hill to the east, the average elevation of the three being about 80 feet above the village streets. The springs are walled up and roofed over, and fairly well protected from surface washings,

though not so well as their surroundings require. Analyses have shown that these springs are not yet seriously polluted.

These springs failed in time to give enough water, and in 1898 three 6-inch wells were drilled 40 feet deep, going 28 feet into sandstone. These wells are in the valley of a small run northwest of the village and are removed from serious contamination. The water rises practically to the surface when the wells are not being pumped, and it is lowered about 15 feet by the pumps.

The water is pumped directly from the wells to the main by a 150,000-gallon horizontal duplex Worthington pump, the surplus going to the spring basins.

There are 6½ miles of mains laid, to which 80 per cent of the population of the village have access. There are 350 services connected, of which 340 are in use, and none is metered. It was impossible to estimate the consumption, as no record has ever been kept either of the flow of the springs or of the amount pumped from the wells.

Waynesburg.—The public water-supply system at Waynesburg (population, 613) was put in by a private company, the Waynesburg Water Company, in 1857, and has cost to date $1,300. The water is obtained by impounding the run-off of two springs located in the upper part of the village, and the supply is carried to the mains by gravity. The reservoir consists of a basin 16 by 33 feet and 5 feet deep, built of brick and covered with a board roof. It has a capacity of about 16,000 gallons and is located at such a height as to give an average pressure of about 20 pounds. The springs emerge from the side of a hill which is used as a pasture. One of them comes from an abandoned coal bank. There has been 0.75 mile of mains laid, giving 40 per cent of the people access to the water; 55 services have been connected, all of which are in use, and none is metered. The daily consumption could not be even estimated.

Waynesville.—On November 1, 1900, Waynesville (population, 723) obtained the approval of the State board of health for a public water supply to be obtained from a number of 6-inch drilled wells located south of the village and near the Little Miami River. The wells are from 40 to 50 feet deep, are drilled through various strata of sand and gravel and into limestone, just above which the water is found.

The wells are practically free from local pollution, and the small amount existing can easily be removed, so that they should furnish an excellent water—much better than the private well supply. Bonds for $30,000 were sold, and a combined electric-light and water plant has just been put in.

The water is pumped to a standpipe having a capacity of 92,000 gallons by two duplex McGowan pumps of 750,000 gallons pumping capacity each. About 3¾ miles of mains have been laid, and 125 services are connected, 123 of which are in use, and none is metered.

West Carrollton.—The public water-supply system of West Carrollton (population, 987) was put in by the village in 1895, and has cost to date $8,500. The supply is obtained from two 8-inch wells drilled in the lowland along the Miami and Erie Canal. The wells are 65 feet deep and go through gravel, hardpan, and into a deep bed of sand, in which the water is found. The normal level of the water is about 10 feet from the surface, and it is lowered to about 30 feet by the highest consumption.

The water is pumped directly to the mains by a horizontal duplex McGowan pump of 750,000 gallons capacity. The average pressure is 60 pounds. The pump is located in the mill of the G. H. Friend Paper and Tablet Company, which has a ten-year contract with the village, under which the company furnishes steam for the pump and keeps it in repair for its use and for the use of the water from the wells. The consumption of water in the village is small, but a large amount is used by the paper mill.

There are 4 miles of mains in, giving about 90 per cent of the people access to the water. There are 128 services connected, of which 120 are in use, and 3 are metered. The well water seems to be of good quality and is used for domestic purposes.

DEEP-SEATED.

Ada.—The village of Ada (population, 2,576) is furnished with water by the Ada Water, Heat and Light Company, which derives its supply from four 8-inch wells, drilled 152 feet deep through 25 feet of clay and soil, 5 to 6 feet of gravel, and the remaining distance in limestone. The wells are located in the eastern part of the village, fairly well removed from local influences. The water is pumped directly to the mains by two horizontal compound duplex Deane pumps of 750,000 gallons capacity each and one Dunn-Gordon of 75,000 gallons capacity. The heating plant has, also, two horizontal compound duplex Laidlaw-Dunn-Gordon pumps of the same capacity, which can also be used on the water supply if necessary. There have been 3 miles of mains laid, to which 75 per cent of the population have access; 150 services have been connected, of which 100 are in use, and 15 are metered. The average daily consumption is 300,000 gallons, which is 116 gallons per capita of the entire population, or 155 gallons per consumer. The supply has just been installed, and the above estimates are therefore not very reliable.

Bluffton.—A public water-supply system was put in by Bluffton (population, 1,783) in 1896 at a cost of $17,000, and has cost to date $24,000. The supply is derived from 8 wells, 8 inches in diameter and from 35 to 50 feet in depth. Only 6 of these wells, however, are now connected with pumps.

The limestone lies within a few feet of the surface, and the water is

obtained in this rock. It rises nearly to the surface when it is not pumped, but can easily be lowered to the suction limit of the pumps. The supply of water has always been sufficient so far, but for use in cases of emergency an 8-inch intake pipe has been run to an old stone quarry near the waterworks. This quarry contains about 3½ acres, and the water is from 3 to 20 feet deep, only the upper 5 feet of which, or 4,000,000 gallons, can be drawn off by the pumps. This emergency intake is infrequently used.

The well water is pumped to an elevated tank by a horiziontal simple duplex Stillwell-Bierce-Smith-Vaile pump of 750,000 gallons capacity. The tank is 20 feet in diameter, 30 feet high, with a capacity of 70,000 gallons, and is placed on a steel-frame tower 82 feet in height, which gives an average pressure of 35 pounds on the main streets.

There have been 4 miles of mains laid, giving 80 per cent of the population access to the water. There are 300 services connected, of which 250 are in use, and only 1 is metered.

The average daily consumption is about 120,000 gallons, or 68 gallons per capita of the population of the village and 81 gallons for each consumer.

Bryan.—A public water-supply system was put in by Bryan (population, 3,131) in 1893–94, and had cost to January, 1899, $45,000. The supply is derived from thirteen wells, eight of which were put down in 1893, and, in the fear that these might not furnish enough water, five more were sunk later. The wells are 4½ inches in diameter, from 95 to 112 feet deep, and go through hardpan, cement, gravel, quicksand, and into loose gravel, where the water is found. When not pumped all the wells flow, and they have always furnished enough water, the level never going below the suction limit of the pumps. The water flows from the wells to a common main, to which the suction pipe is attached, and to which it is directly pumped by two horizontal compound duplex Hughes pumps of 1,000,000 gallons capacity each.

There have been 6 miles of mains laid, which give 75 per cent of the people access to the water. There are 436 services connected, of which only about one-half are in use the year round, the rest being used in summer only for sprinkling. Of these services 55 are metered. The average daily consumption is 176,000 gallons, or 56 gallons per capita of the total population, and 74 gallons per consumer.

Carey.—A public water-supply system was put in by Carey (population, 1,816) in 1894, and has cost to date $30,000. The supply is from a group of five wells west of the village, where the limestone is only 14 feet below the surface. The wells are 5⅝ inches in diameter, from 50 to 60 feet deep, and the water stands normally only 13 feet from the surface, and its level is lowered but 1 foot by ordinary pumping.

The water is pumped directly from the wells to a standpipe by two horizontal compound duplex Laidlaw-Dunn-Gordon pumps of 500,000 gallons capacity each. The standpipe is 16 feet in diameter, 115 feet high, and has a capacity of 173,000 gallons. There are 7 miles of mains laid, which give 80 per cent of the population access to the water.

There are 250 services connected, all of which are in use, and 6 are metered. The average daily consumption is estimated to be 160,000 gallons, or 88 gallons per capita of the total population, and 110 gallons for each consumer.

Carrollton.—A public water-supply system was put in by Carrollton (population, 1,271) in 1895, and has cost to date $66,000. The water is taken from five 8-inch drilled wells, which are 100 feet deep and go through 17 feet of loam and shale and 83 feet into sandstone. The normal level of the water is 8 feet below the surface, and it is lowered to 27 or 28 feet during pumping.

The water is pumped directly from the wells to a standpipe which is 16 feet in diameter and 73 feet high, giving a capacity of 103,000 gallons and a domestic pressure of 60 pounds. The pumps consist of two horizontal compound duplex Worthingtons, of 750,000 gallons capacity each.

There are 4½ miles of mains, giving 85 per cent of the people access to the water. There are 255 services connected, all of which are in use, and 2 are metered. The average daily consumption is 85,000 gallons, which is 67 gallons per capita, and 78 gallons per capita of the population actually supplied. This supply is in general use for domestic purposes.

Continental.—A public water-supply system was put in by Continental (population, 1,104) in 1894, and has cost to date about $10,000. The supply is from three 8-inch wells 75 feet deep, which strike the limestone at a depth of 48 feet. The water is pumped to a standpipe by a deep-well pump of 100,000 gallons capacity. The water rises in the well to within 13 feet of the surface, but the pump cylinder is placed 20 feet down, so that if necessary the water can be lowered about 45 feet; but it has never reached this limit yet, as the well has always supplied enough at a higher level.

The standpipe is 10 feet in diameter, 100 feet high, and has a capacity of 40,000 gallons. For ordinary service the deep-well pump keeps the standpipe full, but in case of fire a 100,000-gallon duplex Stillwell-Bierce pump draws from the standpipe and pumps directly to the main.

There are 2¼ miles of mains in use, which give about 75 per cent of the population access to the water. There are 110 services connected, of which 84 are in use, and 4 of these are metered. The average daily consumption is 45,000 gallons, or 41 gallons per capita of the population of the village, and 154 gallons for each consumer.

Dalton.—A public water-supply system was put in by Dalton (population, 666) in 1895, and has cost to date $8,000. The water is obtained from a well 200 feet deep, drilled in sandstone all the way. The well was drilled 6 inches in diameter for a part of the way, then sent down to the above depth only 4 inches in diameter. The water is pumped by a deep-well pump operated by wind power to a reservoir located at the side of the well, and then flows by gravity to the town to the east. The pump cylinder is 150 feet below the surface, and the water can be lowered to the suction limit below this. The normal level of the water is 40 feet below the surface. The deep-well pump was formerly operated by steam, but owing to the fact that the well is on a high hill, where good wind power can be developed, the change was made. The wind wheel is of the self-adjusting compound pattern, and requires no attention beyond occasional setting and oiling. With this power the pump can handle about 10,000 gallons daily. The reservoir is a circular brick well, covered with a tin roof, and holding 140,000 gallons. In addition to the well supply, the run-off of a spring located in the town has been impounded in a reservoir of 36,000 gallons capacity, which is used only when the well supply is shut off for repairs or for lack of wind.

There are 2½ miles of mains laid, which give 85 per cent of the people access to the water. There are 42 service taps connected, of which 36 are in use, and 8 are metered. Only 3 of the taps are for house use, the others being for sprinkling and stable use only. The average daily consumption is about 3,900 gallons, which is 5 gallons for each person in the village, and 20 for each consumer. This supply is not used at all for domestic purposes.

Delphos.—A public water-supply system was put in by Delphos (population, 4,517) in 1896, and has cost to date $82,000.

The supply is from seven 8-inch wells, ranging from 191 to 300 feet deep, which obtain water from the Onondaga limestone, the top of which is only 20 feet below the surface. Six of the wells flow to a receiving well, and one of them is pumped to this by a Stillwell-Bierce air lift. This one well furnishes nearly all the water, which flows naturally at the rate of 56,000 gallons a day and can be pumped at a rate of 185,000 gallons.

The receiving well is 40 feet in diameter, 37½ feet deep, and has a capacity of 235,000 gallons. It is excavated partly in rock, its sides are lined with brick, and it is roofed over, so that it furnishes an excellent receptacle for a reserve supply of water.

Before the air lift was installed the water was almost unusable from the amount of sulphur present, but the aeration afforded by the lift has overcome this objection, and by the time the water reaches the mains it is free from the taste and smell of sulphur. A strong odor of sulphur is noticeable, however, in the receiving well.

The water is pumped from this well to a standpipe by two horizontal compound duplex Laidlaw-Dunn-Gordon pumps, each of 1,000,000 gallons capacity. The standpipe is 20 feet in diameter, 130 feet high, and has a capacity of 305,000 gallons, which, with the receiving well, gives an ample reserve supply of water.

There are 15.38 miles of mains in use, giving 90 per cent of the population access to the water. There are 615 service taps, of which 600 are in use, and 52 of these are metered. The average daily consumption is 300,000 gallons, which is 66 gallons per capita of the total population, or 74 gallons per consumer.

Lebanon.—A public water-supply system was installed in 1896 by Lebanon (population, 2,867), and has cost to date $52,000. A light plant has recently been established in the same building at a cost of $20,000. The supply is obtained from six 6-inch wells driven from 96 to 104 feet through alternating strata of clay, sand, and gravel. The wells are located a short distance west of the village, in the low ground bordering a small tributary of Turtle Creek.

The water from the wells is siphoned to a receiving well, from which it is pumped to a standpipe by two horizontal compound duplex Stillwell-Bierce and Smith-Vaile pumps of 750,000 gallons capacity each. The supply from the wells is such that the water in the receiving well can be lowered only 10 feet by the present pumping capacity.

The receiving well consists of a brick cistern covered with a tin roof, and is 40 feet in diameter and 25 feet deep, with a capacity of 235,000 gallons. The standpipe is 80 feet high and 25 feet in diameter, with a capacity of 300,000 gallons. The average pressure on the mains is 70 pounds.

There are 12 miles of mains in use, to which at least 95 per cent of the people have access. There are 538 services connected, of which 512 are in use, and 15 of these are metered. Two of the meters are on the lines to the County Infirmary and Children's Home, respectively.

The average daily consumption is 103,000 gallons, or 40 gallons per consumer and 36 gallons per capita of the total population. The water seems to be of excellent quality and is in general use for domestic purposes.

Lewisville.—A water-supply system was installed in 1894 by Lewisville (population, 1,374), and has cost to date $18,000. The supply is obtained from two 8-inch wells, which were drilled 110 feet deep, going through soil and shale into a bed of gravel. The normal level of the water is from 8 to 10 feet below the surface, and it has never been lowered to the suction limit.

The water is pumped directly from the wells to a standpipe by two horizontal duplex Boisard pumps of 500,000 gallons capacity each. The standpipe is 16 feet in diameter and 100 feet high, and has a capacity of 150,000 gallons. The domestic pressure varies from 45 to

50 pounds in the main streets. There are 3½ miles of mains, which give 90 per cent of the people access to the water. There are 165 services connected, of which 159 are in use, and none is metered. The average daily consumption is 40,000 gallons, which is 25 gallons per capita for the village and 42 gallons per capita of the population actually supplied. The water is of good quality, and is used for domestic purposes by those who have access to it.

Madisonville.—A public water-supply system was installed in 1892 by Madisonville (population, 3,140), and has cost to date, together with an electric-light plant, $80,000. The waterworks alone cost about $35,000. The supply is from one 6-inch well and two 8-inch wells, each 150 feet deep and drilled through various strata of clay, quicksand, and gravel down to the rock. The wells are located just west of the village, in the low valley of a branch of Duck Creek. The normal level of the water is 21 feet from the surface.

At first the wells were pumped by direct suction, but this method failed to supply enough water, and in 1895 three deep-well pumps were put in. The wells are only 12 feet apart and within the walls of the pumping station, conditions favorable to the installation of deep-well pumps. All the pumps are the double-acting Laidlaw-Dunn-Gordon type, two having a capacity of 500,000 gallons each and the third 350,000 gallons. The pump cylinders are placed 120 feet from the surface.

The water is pumped by the deep-well pumps to a receiving tank and from this directly to the mains by two horizontal compound duplex Laidlaw-Dunn-Gordon pumps of 1,000,000 gallons capacity each. The receiving chamber consists of a covered steel tank 20 feet in diameter and 15 feet high, with a capacity of 55,000 gallons. The pressure at the station is 80 pounds and the average on the mains is 50 pounds.

There are 10 miles of mains laid, giving 90 per cent of the people access to the water. There are 350 services connected, of which 340 are in use, and 2 are metered. The average daily consumption is estimated to be 110,000 gallons, or 160 gallons per consumer and 35 gallons per capita of the total population. The water seems to be of excellent quality and is in general use for domestic purposes where it is available.

Mansfield.—A public water-supply system was installed in 1870 by Mansfield (population, 17,640), and has cost to date $566,000. The supply is from three groups of wells—one near the pumping station in the northern part of the city, near Rocky Fork; another one-fourth of a mile above this, near the Pennsylvania Railroad, and the third on Bloom street, near the central part of the city.

The station wells were the original supply and were drawn upon until the water in this section was exhausted; then, in 1889 and 1896, the gang one-fourth of a mile above was put in, and in 1896 the Bloom street wells were drilled. In the group near the station there

are ten 4-inch wells and one 10-inch well, which range in depth from 150 to 210 feet and are all in a bed of quicksand. This gang is pumped to a receiving well by a horizontal duplex Blake pump of 750,000 gallons capacity, and supplies about 375,000 gallons daily.

The group one-fourth mile above the station consists of five 5-inch wells and one 10-inch well, all 180 feet deep. These wells penetrate soil and gravel and 60 feet into sandstone rock. The water from them flows to the receiving well by gravity. A centrifugal pumping plant will shortly be installed here, which, it is claimed, will increase the capacity of the group to 2,000,000 gallons daily.

There are two wells in the Bloom street group, each 10 inches in diameter and 180 feet deep. They go through the surface soil, gravel, and clay into a bed of quicksand. The water flows by gravity from these wells directly to the station. The pump well is 35 feet in diameter and 30 feet deep and is walled up with stone and housed over. The water is pumped from this well directly to the mains by two horizontal compound duplex Gordon pumps of 2,000,000 and 1,750,000 gallons capacity, respectively. By closing down the valve in one supply main two pressures are maintained, one of 125 pounds and the other of 80.

There are 60 miles of mains in use, which supply about 70 per cent of the population with water. There are 3,000 services connected, all of which are in use, and 43 are metered. The average daily consumption is about 3,000,000 gallons, which is 170 gallons per capita of the city and 243 for each consumer. The water is in general use by all those able to obtain it.

Montpelier.—A public water-supply system was installed in 1895 by Montpelier (population, 1,869), and had cost to January, 1899, about $44,000. The supply is from three 8-inch drilled wells 113 feet deep, only two of which are now in use. The wells go through clay, fine sand, and into gravel, in which the water is found. The water rises to within 28 feet of the surface; and as the wells were to be pumped by direct suction, the pumps had to be placed 20 feet below the surface, from which place they can lower the water about 45 feet below the surface.

The water is pumped to a standpipe by two horizontal compound duplex Worthington pumps of 750,000 gallons capacity each. The suction pipe is 6 inches in diameter and goes down 36 feet into the well casing.

The standpipe is 12 feet in diameter, 110 feet high, and has a capacity of 93,000 gallons.

There are 4½ miles of mains, which supply about 80 per cent of the people with water. There are 420 services connected, all of which are in use, and none is metered. The average daily consumption is 390,000 gallons, or 209 gallons per capita of the total population and 261 gallons per consumer.

New Bremen.—The village of New Bremen (population, 1,318) will be supplied with water from three 6-inch wells drilled about 100 feet deep through 76 feet of clay and gravel into limestone.

New Comerstown.—On July 22, 1901, a franchise was granted to the New Comerstown Water Company for furnishing a public supply to this village (population, 2,659). The plant was put in operation in the summer of 1902, the supply being obtained from six 6-inch wells driven from 35 to 37 feet deep through clay and loam into a bed of sand and gravel lying along the Tuscarawas River, three-quarters of a mile east of the village.

The water is pumped directly to the mains and reservoir by two horizontal compound duplex Laidlaw-Dunn-Gordon pumps of 1,000,000 gallons capacity each. The reservoir is constructed of concrete or puddle, not covered, and has a capacity of 1,125,000 gallons. It is located on a hill east of the village, at such an elevation as to give a pressure of 107 pounds on the streets. There have been about 6½ miles of mains laid, giving practically the entire population access to the water. It is reported that there are 137 services in use, 6 of which are metered. The average daily consumption is 250,000 gallons, or 85 gallons per capita and 366 gallons per consumer. The industrial use of the water is very large. It is reported that the plant complete cost $149,000, a per capita cost of $56.41. It is hard to see where the money was placed, as this is certainly a large sum to spend for a plant of this character.

New Vienna.—This village (population, 805) has just completed a new waterworks at a cost of $20,000. The supply is obtained from four wells driven 60 feet through gravel and Niagara limestone into the shale that lies below this rock. This water was struck in the southeastern portion of the city by an old gas well. When not pumped the water flows with a small head. A compound duplex John McGowan pump raises the water to the reservoir, which has a capacity of 60,000 gallons.

Four miles of mains have been laid, to which 90 per cent of the population have access. There are 25 services connected, and it is reported that the daily consumption is 2,500 gallons, but the plant is so new that it is impossible to estimate it accurately.

Norwood.—This village (population, 6,480) is on the divide between Duck Creek and Mill Creek, and only about half of its population is tributary to the former, or the Little Miami drainage area.

A public water-supply system was put in by the village in 1894, and has cost to date $154,138. A light plant was installed in the same building, costing, completed, $34,501. The supply was originally derived from six drilled wells, 235 feet deep and 6 and 8 inches in diameter. These wells began to fail and two 10-inch wells were drilled 260 feet deep through clay, gravel, and sand and into the limestone rock a few feet. These have furnished the whole supply since Sep-

tember 1, 1900. The normal level of the water is 110 feet from the surface, and it is lowered but little by pumping. The wells are located in the eastern part of the village, but the water will not be greatly liable to pollution, on account of the depth at which it is found, 240 feet, and the general introduction of sewers, which has prevented the construction of vaults. A third well is being put down so as to insure an ample supply.

The water is pumped to a receiving basin by air lifts and thence to a standpipe by two horizontal compound duplex Laidlaw-Dunn-Gordon pumps of 750,000 gallons capacity each. Compressed air is supplied by a Rand Drill Company's compressor and by a new Ingersoll-Sergeant compressor, the former being held in reserve. A 3-inch air line is carried down 240 feet in the well, and the water is forced up between this and the casing.

The receiving basin is an open oval-shaped cistern, 15 by 20 by 8 feet deep, with a capacity of 14,000 gallons. The standpipe is 60 feet high, 40 feet in diameter, with a capacity of 537,000 gallons, and is placed on a hill of such height that the station pressure is 112 pounds, standpipe full, and the pressure on the mains varies from 104 to 142 pounds.

There are 35 miles of mains laid, giving about 95 per cent of the people access to the water. There are 974 services connected, of which 874 are in use, and 140 of these are metered. Owing to the changes in the plant the consumption could not even be estimated.

Oakwood.—During 1902 the citizens of Oakwood (population, 342) installed a complete water-supply system at a cost of $10,000, or a per capita cost of $40. The supply is obtained from one 6-inch well driven 50 feet into drift, the water being obtained from a stratum of sand and gravel.

The water is pumped to an elevated tank holding 50,000 gallons by a power pump of 350,000 gallons capacity, operated by a gasoline engine; 1.61 miles of mains have been laid and 26 services have been connected, 24 of which are in use, and none is metered. The average daily consumption is 10,000 gallons, or 40 gallons per capita and 91 gallons per consumer.

Orrville.—A public water-supply system was installed in 1894 by Orrville (population, 1,901), and has cost to date $25,000. The supply is derived from four 8-inch wells, which vary from 114 to 121 feet in depth. The wells went through from 10 to 16 feet of soil, then struck the rock, which is a conglomerate of the Waverly group. The water rises above the surface when the wells are not being pumped, and it is lowered only from 10 to 12 feet during the hardest pumping. The supply is pumped directly from the wells to the mains, and a domestic pressure of 70 pounds is maintained by two horizontal compound duplex Laidlaw-Dunn-Gordon pumps of 1,500,000 gallons capacity each. There are 6 miles of mains laid, which furnish 90 per

cent of the people with water. There are 268 services connected, 260 of which are in use, and none is metered. The average daily consumption is 141,858 gallons, which is 71 gallons per capita of the total population and 91 gallons per capita of the population actually supplied. The water is in general use and is of good quality, both for domestic and manufacturing uses. A good deal of water is used by the railroads that pass through the town.

Paulding.—A public water-supply system has just been installed at Paulding (population, 2,080) at a cost of $38,000. The supply is taken from three 8-inch wells from 320 to 616 feet deep. The water is obtained from the limestone found just below the drift. The wells are located in the northern part of the village, at a point free from local pollution, and the rock is well protected by about 25 feet of clay. The water is pumped from the wells to a reservoir by two horizontal compound duplex McGowan pumps of 750,000 gallons capacity each.

Four miles of mains have been laid, to which 70 per cent of the population have access. There are 95 services connected, all of which are in use, and none is metered. The average daily consumption is 80,000 gallons, or 38 gallons per capita of the total population, or 55 gallons per consumer.

Perrysville.—A public water-supply system was installed in 1894 by Perrysville (population, 513), and has cost to date $8,000. The supply is derived from one 6-inch well 93 feet deep, 53 feet of which is sandstone rock. The well is located just north of the village, far away from all source of pollution. The water is pumped to a small reservoir by a Smith-Vaile deep-well pump of 112,000 gallons capacity. The normal level of the water is 25 feet below the surface, and the well can be pumped dry, as the water cylinder is 75 feet down.

The reservoir, which is of 54,000 gallons capacity, consists of a brick cistern covered with a shingle roof, and is located on a hill back of the pumping station at such height as to give a pressure of 75 pounds in the village.

There are 3 miles of mains in use, supplying 75 per cent of the people with water. There are 60 services connected, 45 of which are in use, and 25 are metered. The average daily consumption is 12,600 gallons, which is 25 gallons per capita of the village and 35 per consumer. The water is in very general use in the village, to the exclusion of private wells. The absence of baths and water-closets keeps the consumption very low.

Rockford.—The public water-supply system of Rockford (population, 1,207) was put in by the village in 1897, and has cost to date about $17,000, which is a per capita cost of $14.08.

A private company owns and operates an electric-light plant in the waterworks pumping station and obtains its power from the waterworks boilers. This arrangement, it is said, pays the waterworks enough annually to cover the cost of pumping the entire supply for the village.

The supply is obtained from two 6-inch wells, drilled 109 and 111 feet, respectively, into an extensive bed of gravel. The normal level of the water is from 11 to 16 feet from the surface, and it is lowered but little in one and some 50 feet in the other by ordinary pumping.

These wells are pumped by two Laidlaw-Dunn-Gordon deep-well pumps of 150,000 gallons capacity each, which discharge into a receiving well, from which the water is pumped to a standpipe by a horizontal compound duplex Laidlaw-Dunn-Gordon pump of 500,000 gallons capacity.

The receiving well is 24 feet in diameter and 14 feet deep and has a capacity of 50,000 gallons. It is constructed of brick and covered with a tin roof. The standpipe is 107 feet high and 12 feet in diameter, has a capacity of 94,000 gallons, and furnishes an average pressure on the mains of 40 pounds.

There are 3½ miles of mains laid, which give about 90 per cent of the people access to the water. There are 52 services connected, all of which are in use, and 10 are metered. The average daily consumption is about 23,000 gallons, which is 442 per service, 38 per consumer, and 19 per capita of the village's population.

The wells are located in the northwestern part of the village, far away from any source of serious pollution, and the water seems to be of good quality.

There are a number of private wells still in use. These are usually dug about 35 feet deep, going through clay and hardpan into the gravel below.

St. Marys.—In 1895 a public water-supply system was put in by St. Marys (population, 5,359), and had cost to December, 1898, about $57,500. The supply is derived from four 8-inch drilled wells, which range from 262 to 272 feet in depth. The wells strike the limestone at 125 feet, and from this stratum obtain a very hard water, which contains some sulphur and iron.

The wells flow to a covered receiving well, 38 feet in diameter and 28 feet deep, that holds 237,000 gallons. From this well the water is pumped directly to the mains by two horizontal compound duplex Smith-Vaile pumps, each of 750,000 gallons capacity.

There are 12½ miles of mains laid, which give 75 per cent of the population access to the water. There are 280 services connected, of which 275 are in use, and 7 of these are metered. The average daily consumption is about 100,000 gallons, or 28 gallons per capita of the population of the village and 60 gallons per consumer.

Scio.—The water-supply system at Scio (population, 1,214) was put in by a private company, the Scio Water Supply Company, in 1899, and has cost to date about $35,000. The plant was originally designed to protect the village against the great danger from fire on account of the oil industry. The water is obtained from five 8-inch wells 105 feet deep. Four of the wells go through quicksand into sandstone rock, and one goes through rock into a broken stratum of rock and

clay. The normal level of the water is 2 feet below the surface, and it is lowered but little by pumping. The wells are located in the valley of Conotton Creek, just below the village, and would be badly polluted by drainage if they were not carefully cased in. The water is pumped directly from the wells to two elevated tanks by a Smith-Vaile triplex double-action pump of 1,000,000 gallons capacity. The tanks are constructed of steel, each 18 feet in diameter and 25 feet high, and have a combined capacity of 95,000 gallons. They are placed on a hill back of the pumping station and at such height as to give an average pressure of 100 pounds.

There are 4 miles of mains, which give 95 per cent of the people access to the water. There are 120 services connected, and 110 are in use, of which 1 is metered. The average daily consumption is 31,500 gallons, which is 17 gallons per capita of the village and 150 gallons per consumer.

Shreve.—The public water-suply system of Shreve (population, 1,043) was put in by the village in 1893, and has cost to date $11,000. The water is obtained from an 8-inch well 142 feet deep, located in the upper part of the village. The rock is very near the surface at this point, and the well required but little casing. The water is pumped by a Laidlaw-Dunn-Gordon deep-well pump, of 100,000 gallons capacity, to a small brick cistern, from which it is pumped to a reservoir by a horizontal duplex Miller pump of 150,000 gallons capacity. The cistern holds 3,300 gallons and the reservoir 88,200 gallons. The reservoir, which is constructed of brick and is covered over, is 30 feet in diameter and 16 feet deep. It is located on a hill north of the village, at such height as to give a pressure of 80 pounds. It is cleaned every year, and no trouble has been given by vegetable growths.

There are 2½ miles of mains laid, which supply 90 per cent of the people with water. There are 122 services connected, all of which are in use, and all are metered. The average daily consumption is about 54,000 gallons, which is 51 gallons per capita of the total population and 106 gallons for each consumer. The water is of good quality and is used for domestic purposes by everyone having access to it.

Sidney.—Waterworks were first established by Sidney (population, 5,688) in 1874. The old plant cost $50,000 and the new one, put in in 1889, has cost to date $107,500. The first source of supply was Mosquito Creek. This was abandoned in 1889, and water was then taken from Miami River. The water was filtered through two American pressure filters of 225,000 gallons capacity each, and later a New York horizontal pressure filter of 500,000 gallons capacity was put in. In 1895 wells were drilled and water enough to supply the town was found. Now these form the sole source of supply, and the creek water is not used at all, though the intake is intact and the water could be drawn on in case of emergency. The filters were also put out of use as soon as the well supply was developed. The present supply is

derived from six wells, 120 feet deep and 8 inches in diameter, located near the pumping station. The wells are all artesian; they flow at the rate of 1,000,000 gallons in twenty-four hours during favorable seasons, and are lowered only 10 feet by the pumps. They go through a bed of gravel 20 feet thick and then strike the Niagara limestone, in which they find water which is of excellent quality except for its hardness. The water is pumped directly from the wells to a standpipe situated on a hill back of town. The two pumps in use, put in in 1889, are vertical compound duplex Gordon pumps of 1,500,000 gallons capacity. The standpipe is 25 feet in diameter, 86 feet high, and has a capacity of 300,000 gallons. It gives a pressure of 70 pounds at the station.

There are 13 miles of mains laid, giving about 90 per cent of the population access to the water. There are 700 services connected, of which 685 are in use, and 60 of these are metered. The average daily consumption is 639,000 gallons and the average per consumer is 183 gallons, or 112 per capita.

Sycamore.—The public water-supply system of Sycamore (population, 853) was put in by the village in 1898–99, and has cost to date, together with the electric-light plant, $25,000.

The supply is obtained from two 6-inch wells, one 204 and one 198 feet deep, located at the eastern edge of the village. The wells go through 100 feet of drift and then strike the limestone bed rock, in which the water is found. The normal level of the water is about 23 feet below the surface of the ground, and it is lowered but little by the severest pumping.

At first the whole supply was obtained from these wells by a Snider-Hughes air lift, but in June, 1900, a Hill deep-well pump of 75,000 gallons capacity was installed in one well for fear of the failure of the air lift. Both the air lift and the deep-well pump discharge to a covered brick cistern of 40,000 gallons capacity, from which the water is pumped to an elevated tank by a horizontal compound duplex Snider-Hughes pump of 500,000 gallons capacity. The water tower consists of a 40,000-gallon steel tank on a steel tower 80 feet high, giving a total height of 100 feet.

About 7 miles of mains have been laid, giving about 80 per cent of the people access to the water. There are 250 services connected, of which 140 are in use, and one of these is metered for the supply of a local railroad. The average daily consumption is 90,000 gallons, which is 105 gallons per capita, or 132 gallons per consumer.

Van Wert.—The public water-supply system of Van Wert (population, 6,422) was put in by the village in 1891, and has cost to date $130,000. The supply is from nine 6-inch wells 200 feet deep, which strike the limestone in which the water is found at 40 feet. The water rises to within from 12 to 18 feet of the surface and is pumped down about 40 feet, the pumps being set 20 feet below the surface.

The water is pumped directly from the wells to the mains by two

horizontal compound duplex Holly pumps of 1,500,000 gallons capacity each. An open reservoir, 165 feet square and 18 feet deep, holding 3,000,000 gallons, is used to store water for an emergency. It is kept full all the time and is drawn on by the pumps only in case of fire, when the wells would be exhausted by so sudden an increase in consumption.

There are 19 miles of mains in use, which give about 75 per cent of the population access to the water. There are 1,035 services connected, of which 650 are in use, and 205 of these are metered. The average daily consumption is 300,000 gallons, or 47 gallons per capita of the total population, or 62 gallons per consumer.

Wauseon.—The public water-supply system of Wauseon (population, 2,148) was installed by the village in 1897, was first operated in April, 1898, and had cost to January, 1899, $25,000. The supply is from one of two 8-inch wells 185 feet deep, which are sunk through shale and fine gravel into coarse gravel, in which the water is found. The water comes to within 30 feet of the surface, and is pumped to a receiving well by a 400,000-gallon Downey deep-well pump, so set up as to be able to lower the water to 75 feet from the surface if necessary. During ordinary pumping the well not in use, which is 150 feet away, is lowered 6 feet. The receiving well, which is covered with a tin roof, is 19 feet deep, 30 feet in diameter, and has a capacity of 100,000 gallons. The water is pumped from this basin to a standpipe by a horizontal compound duplex Smith-Vaile pump of 750,000 gallons capacity. The standpipe is 12 feet in diameter, 120 feet high, and has a capacity of 100,000 gallons.

There are 7½ miles of mains laid, giving 90 per cent of the population access to the water. There are 301 services in use, of which 275 are metered. The average daily consumption is 45,000 gallons, or 20 gallons per capita of the total population, or 60 gallons per consumer.

West Alexandria.—The public water-supply system of West Alexandria (population, 740) was installed in 1897, and has cost to date $25,643.65. The supply is obtained from four artesian wells located in the southwestern part of the village. The wells are 6 inches in diameter; two of them are 132 feet deep, one is 95 feet deep, and one is 65 feet deep, and all go through alternate layers of clay, gravel, and hardpan. It is claimed that the wells will flow with a head of 16 feet above the surface.

Only the natural flow of the wells is utilized, the water being conducted to a receiving basin, from which it is pumped to the standpipe by two horizontal duplex Smith-Vaile pumps of 1,000,000 gallons capacity each. The basin consists of a covered brick well, 20 feet in diameter, 24 feet deep, having a capacity of 7,500 gallons. The standpipe consists of a steel tank 40 feet high and 10 feet in diameter, holding 60,000 gallons, placed on a brick tower 60 feet high. The average pressure on the mains is 40 pounds.

There are 3 miles of mains in, to which all of the people have access. There are 131 services in, all of which are in use, and all are metered. The average daily consumption is estimated to be 26,767 gallons, or 36 gallons per consumer, and 36 gallons per capita of the village's population.

SUBSURFACE AND DEEP-SEATED.

Hicksville.—The public water-supply system of Hicksville (population, 2,520) was put in by the village in 1890, and has cost to date about $25,000. The supply is obtained from seven wells, four of which are 32 feet deep and 6 inches in diameter, two are 132 feet deep and 6 inches in diameter, and one is 132 feet deep and 3 inches in diameter.

The wells go through a mixture of clay and sand, the deeper ones reaching a layer of sand and gravel found just above the Ohio black shale, 130 feet down. All the wells flow with a head of 4 feet when not pumped, but do not supply enough water in dry season without pumping. The water from six of the wells is pumped directly to the mains by two horizontal duplex Hughes pumps of 750,000 gallons capacity each. The flow of the seventh well goes to a small reservoir, 100 feet square, holding 387,000 gallons, which is kept on hand as a reserve in case of fire.

There are 4.5 miles of mains in use, which give about 75 per cent of the population access to the water. There are 290 services connected, of which 260 are in use, and none is metered. The average daily consumption is 90,000 gallons, or 36 gallons per capita of the total population and 47 gallons for each consumer.

Wapakoneta.—A public water-supply system was put in by Wapakoneta (population, 3,915) in November, 1895, and had cost to April 1, 1903, $63,109.76. The supply is from nine 6-inch drilled wells, eight 35 feet deep and one 142 feet deep, and from a large dug well 30 feet deep and 20 feet in diameter. Only one of the shallow wells is pumped. The water collected by the other shallow wells now flows through tile directly to the large well, which acts as a collecting well and also as a pumping well. The shallow wells obtain their water from a bed of gravel found at the level of Auglaize River. The deep well goes through clay and gravel into limestone, which is found at a depth of 135 feet. The water is pumped directly to the mains in summer and to a standpipe in winter by two horizontal compound duplex McGowan pumps of 1,000,000 gallons capacity each. The standpipe is 125 feet high and 25 feet in diameter, and has a capacity of 459,000 gallons.

There are 12.75 miles of mains in, giving about 90 per cent of the population access to the water. There are 475 services connected, of which 469 are in use, and only 15 are metered. The average daily consumption is 198,000 gallons, which is 51 gallons per capita of the total population, or 56 gallons per consumer.

SURFACE AND GROUND.

SURFACE AND SUBSURFACE.

Blanchester.—The public water-supply system of Blanchester (population 1,788) was put in by the village in 1896, and has cost to date $34,000. An electric-light plant costing $10,000 is in the same building. The supply was to be obtained from three wells 6 feet in diameter and 50 feet deep, but they soon failed, and the village was without water, as there were no funds available for a new supply. In 1899 a private company was formed among the citizens of the village for the purpose of securing a supply of water for fire protection at least. A franchise was obtained from the village, under which the company was to supply the water, the village to provide pumps and pay the cost of pumping, and the company to have the privilege of collecting the private water rents.

As ground water was not available and there were no streams nearby of sufficient size to afford a supply, a reservoir was excavated in the valley of a small run having a drainage area of 1,400 acres, and a surface supply was thus impounded. The reservoir has an area of about 3 acres and an average depth of 6 feet, giving a capacity, when full, of about 5,000,000 gallons. The sides and bottom of the reservoir are not paved or protected in any way, and the water is very muddy for a long period after each rain. In the spring of 1900 a heavy rain caused a flood, too large for the wasteway, and the reservoir banks were overflowed and a portion was broken out. The wasteway is 10 feet long and can carry a flow 5 feet deep before the reservoir banks will overflow. This would require a rainfall of $\frac{1}{4}$ inch per hour over the entire watershed of such duration as would allow the water from the most remote section to reach the river; it would also have to fall on a soil that would be impervious from continual rain or freezing. The reservoir, with connections, cost $6,000.

The water from the reservoir flows by gravity through an 8-inch pipe to the old wells, in which the suction of the pumps is placed. The water in the wells will rise to within 7 feet of the surface when pumping is suspended for some time, and the reservoir line will not flow until the wells have been lowered to 12 feet, so that a small amount of well water is supplied, but it is a very small proportion of the total consumption and can hardly be considered.

The water is pumped directly from the suction wells to an elevated tank by two horizontal compound duplex McGowan pumps of 750,000 gallons capacity each. The pressure is 50 pounds at the station and averages 45 pounds on the mains.

There are 11 miles of mains laid, to which 90 per cent of the people have access. There are 150 services connected, 140 of which are in use, and 12 are metered. The average daily consumption is about 15,000

gallons, which is 333⅓ gallons per service, 35 gallons per consumer, and 28 gallons per capita of the village.

The water is not used for domestic purposes at all, and is not fit for such use without filtration.

Lima.—The public water-supply system of Lima (population, 21,723) was put in by the city in 1885–1887, at a cost of $387,000. Many extensions and additions have been made since that time, and the cost to date is much more than the sum stated. The original supply was from an impounding reservoir on Lost Creek, a small stream that enters the Ottawa River just east of Lima. A timber dam, 10 feet high and 30 feet long, was built across the creek, forming a reservoir 6 acres in area and having a capacity of about 10,000,000 gallons. This reservoir is dirty, overgrown with weeds, very shallow in many places, and is dry all summer except for a few stagnant pools.

The winter and spring rains fill the reservoir and the water is run to a receiving well, from which it flows by gravity to two storage reservoirs, situated side by side, and having a combined area of 21 acres, a depth of 17 feet, and a capacity of 120,000,000 gallons. The sides and bottom of these basins are of natural clay, but no trouble has been experienced from vegetable growths, except during one year, though they have been cleaned but once. Formerly these reservoirs had to be filled during freshets or hard rains, and the water was then supplied to the city until another rain came. If the water was low, the first part of the rain had to be caught and used, and the first washings of the small drainage area were especially foul, as they brought in the waste from a number of oil wells within the area. There was soon considerable complaint about the water, and in 1894 eight wells were put down at the head of the impounding reservoir in an endeavor to find ground water enough to supply the city.

The wells are all 8 inches in diameter and are from 150 to 200 feet deep. They are pumped to the receiving well by a 1,000,000-gallon pump operated by an electric motor furnished with power from the central station. The water in the wells is lowered below the suction limit in a short time by pumping, as they are placed too close together.

In 1897 a 10-inch well was put down north of this group and an abandoned oil well also was appropriated. These are pumped to the receiving well by machinery similar to that used on the first group. It is estimated that all the wells supply 1,250,000 gallons a day, which allows better water to be obtained from the surface source.

The water is pumped from the storage reservoir directly to the mains by two horizontal compound duplex Holly pumps of 3,000,000 gallons capacity each. There are 45 miles of mains laid, giving about 80 per cent of the population access to the water. There are 3,705 connections in use, of which 300 are metered. The average

daily consumption is 1,624,805 gallons, which is 61 gallons per capita of the total population, or 120 gallons for each consumer.

Since the above information was obtained Lima has had more trouble from shortage of water and further additions will probably be made to the ground supply.

Newark.—The public water-supply system of Newark (population, 18,157) was put in by a private company, the Newark Water Company, in 1886, and has cost to date about $250,000. The water was first obtained from two brick-lined wells, 21 feet deep and 20 feet in diameter, dug in a gravel bed about a mile north of the city, near the north branch of Licking Creek. Later these wells were supplemented by twenty-four 4-inch Cooke driven wells located at the same place. The supply still being short, a small brush-and-stone dam was put in the creek and the water was thus diverted to one of the large wells, which serves as a pump well. All the wells are in sand and gravel, the deeper ones just reaching a layer of hardpan below this bed. The normal level of the water is about 10 feet below the surface and it is lowered 4 or 5 feet by pumping.

The pumps are attached directly to the Cooke wells and to one of the large wells, the other flowing to this one through a siphon. The water is pumped to an open reservoir by two horizontal compound duplex Gordon-Maxwell pumps of 2,500,000 gallons capacity each. The reservoir is about 250 by 60 feet, and 12 feet deep, with an available capacity of 850,000 gallons. It is located on a hill across the creek from the pumping station, at such a height as to give a pressure of 114 pounds. It is excavated in the top of the hill, and the sides are formed of earthen embankments, puddled and roughly paved. No trouble has been experienced from vegetable growths.

There are 27 miles of mains laid, to which 80 per cent of the people have access. There are 1,900 services connected, of which 1,600 are in use, and 450 are metered. The average daily consumption is 1,545,000 gallons, which is 85 gallons per capita of the city and 105 gallons for each consumer. The water is not in general use in the city, but its quality is good, much better than that of private well water. The surface water is used only when the water of the creek is clear, and in this way is not very objectionable, though the run-off is derived from a thickly settled farming district and is rather impure. The large average daily consumption per consumer is due to the use of the water by one of the railroads.

Springfield.—Waterworks were put in by Springfield (population, 38,253) in 1881, and have cost to date $666,402. The first supply was from a filter gallery. This soon failed, and two years later a reservoir was built just above the junction of Beaver and Buck creeks, at the site of the present pumping station. This reservoir is 350 feet long, 200 feet wide, and 18 feet deep, giving a capacity of about 8,000,000 gallons. The bottom and sides are only graveled and are badly

covered with weeds and moss. The reservoir is filled by the seeping in of ground water. In order to raise the level of the ground water and keep the reservoir full, a dam was built about 300 feet below the junction of the creeks. This dam is 50 feet long and 4 feet high, and backs the water up past the reservoir. This reservoir and the old gallery supplied the city until 1894–95, when the new station and gallery were put in and the old station was abandoned. The new gallery is 200 feet long, 32 inches wide, and 48 inches high, and lies 20 feet below the surface, in an extensive gravel bed, which is situated under and between the two creeks at their junction. The gallery leads to a covered pump well 30 feet in diameter and 21 feet deep. This gallery supplies an excellent water and, so far, plenty of it. The reservoir is connected to the pump well by a 36-inch main, to be used in cases of emergency only.

The old station contains a 2,500,000-gallon Cope and Maxwell pump that can be used to pump the water from the old gallery to the new pump well in case of failure of the new supply due to continued drought. The new station contains a horizontal compound duplex Holly pump of 5,000,000 gallons capacity and a vertical triple-expansion Allis pump of 7,000,000 gallons capacity. The Holly pump was brought from the old station, but has been remodeled, and is in good condition now. The Allis pump is a new one, put in in 1895. The water is pumped to a standpipe 30 feet in diameter and 112 feet high, having a capacity of 592,000 gallons.

On account of the extremely dry weather of 1899 it was necessary to use water from the wells and galleries of the old station, the former source of the total supply. The water from this source was pumped by the engine left at the old station to the reservoir near the new station and allowed to flow from there to the pump well. Even this increase in the supply was not sufficient for all purposes, and Springfield was left without adequate protection from fire.

To overcome this danger the trustees, in the early part of the summer of 1900, tapped the old 20-inch main leading from the old station to the reservoir at Buck Creek and put in a 12-inch pipe, so that the creek water can flow directly to the reservoir. The intention was to keep the reservoir full in this way and to allow the water to seep through into the gallery and pump well and only use the water directly in case of fire. This reservoir is 250 by 500 by 15 feet deep, and as it is excavated in a rather porous gravel bed, normally its water level is equal to and varies with the height of the ground water.

It was thus the intention to reenforce the ground water with seepage from this reservoir supplied from the creek, and such use would not be objectionable; but as the reservoir puddled very soon with turbid creek water, this seepage was slow, and it was necessary on a few occasions to run the creek water directly to the pump well by way of the reservoir and a 36-inch pipe from the reservoir to the pump well.

There are local as well as sanitary difficulties regarding the use of Buck Creek water, in the nature of suits for diverting the water from the mills below, etc.

There are now 66 miles of mains in use, giving some 90 per cent of the people access to the water. There are 5,000 services in, of which 5,000 are in use, and 280 of these are metered. The average daily consumption is 3,250,000 gallons, which is 194 per consumer, and 85 per capita of the total population.

Wooster.—A public water-supply system was installed in 1875 by Wooster (population, 6,063), and has cost to date about $280,000. The first water was supplied by gravity from the Redick reservoir, which is formed by a dam across Christmas Run, just north of Wooster. The run above the dam has a drainage area of 8 square miles, on which are located a number of farm houses and the County Children's Home, an institution with 64 inmates and officers, part of whom use closets sewered direct to the run, at a point only three-fourths of a mile above the reservoir. This reservoir will hold 8,000,000 gallons, with an average depth of 18 feet. Its sides are roughly paved, but no provision is made to prevent soil and clay from washing in from its slopes. On account of the small drainage area, it is necessary to fill the reservoir during freshets, when the water is at its worst stage. A few springs send a small quantity of excellent water to this reservoir.

Need was soon felt for a better and more abundant supply and also greater pressure, for the reservoir is only 114 feet above the streets. In 1881 a well 47 feet deep and 33 feet in diameter was dug northeast of the village and a quantity of good water was obtained; not enough, however, to allow the Redick reservoir to be abandoned; so about one-half of the people must still use the old supply.

The water from the well is pumped by a 750,000-gallon horizontal duplex Worthington pump directly to a new reservoir located on high land to the north. When not being pumped the well contains 13 feet of water, which is pumped out in about five hours, and then the well is allowed to fill again.

The reservoir is circular, 1 acre in area, and holds 4,000,000 gallons when filled to a depth of 13 feet. Its sides are carefully paved and no surface drainage can enter it. It is 160 feet above the pumping station and 248 feet above the main streets, and so furnishes ample pressure. This part of the supply would be without objection if the ground water were not exposed in an open reservoir where growths are developed which give it a very bad odor and taste.

Combined, these supplies have failed during dry seasons, and it has been necessary to use water from Apple Creek. A horizontal duplex Worthington pump, with a capacity of 500,000 gallons an hour, is used to pump the creek water directly to the mains in the dry summer months.

There are 11.75 miles of mains laid, to which 80 per cent of the people have access. There are 487 services connected, of which 390

are in use, and 47 are metered. The average daily consumption is about 700,000 gallons, half of which is supplied from each reservoir. The average daily consumption per capita of the total population is 115 gallons, and the per capita of the population actually supplied is 142 gallons. Very little water is used for domestic purposes, most of the above quantity being used for sprinkling, water-closets, and industrial purposes.

Xenia.—The public water-supply system of Xenia (population, 8,696) was put in by a private company in 1887 and has cost to date, as claimed, $203,000. The company now operates under the name of The Xenia Water Company.

The first supply was obtained by impounding the run-off of a number of springs located 1½ miles north of the city. The flow from these is collected in an open reservoir, formed by an earthen embankment, with sheet-piling core wall, thrown across a small valley and forming a basin 1½ acres in area. Through the center of the reservoir there is a dividing wall for convenience in cleaning. This, as well as the inner side of the dam, is roughly paved. Much of the surface drainage to the reservoir is diverted, but some of it is used. Later, near this reservoir, a large well, 75 feet square, was dug 25 feet deep through a bed of clay and hardpan into gravel. The water from this is pumped by a 1,000,000-gallon horizontal duplex Worthington pump to a large stone pump well, 40 feet in diameter, built in the dividing wall of the reservoir. The combined water of the well and reservoir is then pumped through the mains to a standpipe by two horizontal compound duplex Gordon-Maxwell pumps of 1,500,000 gallons capacity each. It is estimated that the springs will supply 300,000 gallons daily and the well 100,000.

In 1896 these sources were supplemented by an additional ground supply secured from six 6-inch wells and three 8-inch, located southwest of the city, just out of the corporation limits. These wells are from 28 to 40 feet deep, and go through a bed of clay and into gravel. The normal level of the water is 6 to 8 feet from the surface, and it can easily be lowered to the suction limit, indicating a rather weak vein at the above depths. A test well is now being put down into the rock, which is found at 60 feet, in the hope of securing a more abundant supply.

The water from this last set of wells is pumped by a 1,000,000-gallon horizontal compound duplex Deane pump directly to the mains and standpipe. This station supplies about 175,000 gallons a day for a few days each week in summer and fall, and is not used in winter.

The standpipe is 115 feet high by 20 feet in diameter, with a capacity of 270,000 gallons, and furnishes, when full, an average pressure on the mains of 55 pounds.

There are 20 miles of mains laid, to which almost all of the people have access. There are 1,200 services in, of which 1,100 are in use,

and 50 of these are metered. The average daily consumption is 500,000 gallons, which is 57.7 gallons per consumer, or 57.5 gallons per capita of the total population.

SURFACE AND DEEP-SEATED.

Canton.—The public water supply of Canton (population, 30,667) was put in by the city in 1869, and has cost to date about $800,000. The first supply was taken from the west branch of the Nimishillen Creek, which was the sole source until 1888, when the supply was increased and improved by the addition of water from a number of wells.

The water from the creek is diverted to a race by a small dam 35 feet long and 1 foot high, located 1.3 miles above the pumping station. The western branch has above the dam a drainage basin 48 square miles in area, on which there are several small villages and one public institution. This last, the Stark County Infirmary, is located just above Canton, and sends its sewage to a gravel swale, from which some of it may be washed into the creek by heavy rains. A public ford just above the dam is also the source of some objectionable pollution. This surface water is one of the few in Ohio that is highly colored at times from organic matter.

In 1888 26 wells were put down in the park adjoining the pumping station, and these have been added to until there are now 36 in all. These wells vary from 4 to 8 inches in diameter and from 225 to 365 feet in depth. The water is possibly obtained from a bed of sandstone, into which most of the wells go after passing through an irregular stratum of limestone. The normal level of the water is near the surface, but is easily lowered from 25 to 30 feet by direct pumping.

The wells are pumped by a 2,000,000-gallon Gordon pump to an open well, into which the water from the race flows by gravity. The mixed waters are then pumped directly to the mains by two horizontal compound duplex Worthington pumps of 2,000,000 gallons capacity each, and one horizontal compound duplex Holly pump of 8,000,000 gallons capacity.

The average domestic pressure maintained is 80 pounds. There are 65 miles of mains in use, giving 60 per cent of the people access to the water. There are 5,200 services connected, of which 4,700 are in use, and 198 of these are metered. The average daily consumption is 4,000,000 gallons, of which about 1,200,000 gallons is supplied by the wells and the remainder directly from the creek. A number of new wells are now being put in and the amount of surface water used will probably soon be decreased. The average daily consumption per capita of the population is 130 gallons and the per capita of those actually supplied 217 gallons.

Massillon.—This city (population, 11,944) has been supplied with water from various sources since 1854, and always by private com-

panies. In 1881 the run-off of Sippo Lake and Sippo Creek was impounded and the city was supplied entirely with this water. The quality was not satisfactory, and in 1893 a new supply for domestic use was procured from six drilled wells. The old surface supply is used for industrial purposes and street sprinkling. It is furnished by gravity from a reservoir having a capacity of about 30,000,000 gallons.

The wells from which the domestic supply is procured are located in the river valley, in the upper part of the city. They are 6 inches in diameter and average 200 feet in depth, going through clay and gravel into sandstone. Four of the wells flow when not pumped, and in the other two the water rises very near the surface; average pumping lowers the level about 15 feet. The water is pumped directly from the wells to a standpipe by three pumps—a horizontal compound duplex Blake of 2,000,000 gallons capacity, installed in 1887 at the old station and removed in 1893 to the present one; a horizontal compound duplex Deane pump of 1,500,000 gallons capacity, and a horizontal compound duplex Worthington pump of 500,000 gallons capacity, the last two installed in 1893.

A small covered reservoir 18 feet deep, 30 feet in diameter, and holding 100,000 gallons is used as a reserve supply for the wells. The standpipe was erected in 1883 for use on the surface supply. It is 150 feet high, 25 feet in diameter, and has a capacity of 550,000 gallons. A horizontal compound duplex Blake pump of 2,000,000 gallons capacity stands in the old pumping station at the reservoir and can be used to pump this water to the standpipe in cases of emergency.

The average pressure for the gravity supply is 26 pounds; that for the domestic supply is 90 pounds. There are 2 miles of mains laid for the surface water and 24 miles laid for the well water. About 90 per cent of the total population are within reach of the domestic service. There are 1,300 services in, all of which are in use, and 75 are metered. The average daily consumption for the domestic supply is only 650,000 gallons, or 54 gallons per capita of the total population and 60 gallons per capita of the population actually supplied. These figures are of special interest, as they represent the amount of water actually used for the domestic supply of a city. The amount of water used on the gravity supply could not be obtained.

Tiffin.—The waterworks at Tiffin (population, 10,989) were put in by a private company in 1879. The cost of the plant could not be learned. The supply was originally taken entirely from Sandusky River, but this source was supplemented later with water from a number of wells. The river water is badly polluted with the sewage of Crestline, Bucyrus, and Upper Sandusky, the latter village being 34 miles above.

The waterworks are at the upper end of the village, and 400 yards above this is a dam that diverts the water to a small reservoir, from

which it is pumped, and to a race which furnishes power for a portion of the time. The dam is constructed of timber, is 175 feet long and 5½ feet high, and backs the water 4 miles up the river.

There are seven drilled wells, each 10 inches in diameter and from 200 to 268 feet deep, all in the Onondaga limestone, which outcrops in this locality. The wells extend in a line up and down the river valley and are from 200 to 300 feet apart. The pump suction is connected directly to the well casing, and the water is lowered about 24 feet by ordinary pumping. It is claimed that these wells supply all the water except in cases of emergency, and that a better method of pumping will be put in, so that no river water will be needed. These claims are hardly realized, however, as the river water is used daily.

Direct pressure is furnished by three Holly pumps. One, put in in 1879, is run by water power and has a capacity of 1,500,000 gallons; the other two are steam pumps, put in in 1882 and 1895, and have capacities of 3,000,000 and 4,000,000 gallons.

There are 26 miles of mains, which give about 90 per cent of the people access to the water. There are 1,300 services connected, of which only 900 are in use, and 175 are metered. The average daily consumption is estimated at 1,000,000 gallons, being 91 gallons per capita of the total population, or 101 gallons for each consumer. The supply is not satisfactory owing to the constant use of the river water.

SURFACE, SUBSURFACE, AND DEEP-SEATED.

Fremont.—A public water-supply system was put in by Fremont (population 8,409) in 1883, and had cost to July, 1898, $203,766.96. The supply is obtained from Sandusky River at a point above the city, from wells, and from galleries. Above the place where the water is taken from the river it receives the sewage of Crestline, Bucyrus, Upper Sandusky, and Tiffin, the latter only 21 miles above.

The river water is admitted through three 10-inch pipes to a reservoir excavated in the solid rock below the river level. This basin is also supplied to some extent by three lines of vitrified tile, 6 to 9 inches in diameter, having a total length of 1½ miles, which are laid in gravel beds along the river bottom. The reservoir is 200 feet long by 150 feet wide, and the water stands from 6 to 8 feet deep in it, or at the river level, giving a capacity of from 1,400,000 to 1,800,000 gallons. Large quantities of sand and mud collect in the reservoir, which have to be removed by buckets, as the bottom is too low for flushing.

Five 10-inch wells, 300 feet deep, were drilled in 1897. These are pumped by airlifts and supply about 230,000 gallons daily. The machinery for pumping these wells is inadequate and it is intended to install better, so as to increase the supply, though at present the water is lowered from 40 to 125 feet in the wells.

The water is pumped to a standpipe by two Knowles pumps, one a 3,000,000-gallon compound duplex, and the other a 1,000,000-gallon

duplex. The standpipe is located in the center of the village and is 100 feet high, 25 feet in diameter, and has a capacity of 365,000 gallons. There are 23 miles of mains in use, which give 75 per cent of the total population access to the water. There are 1,115 services connected, of which 905 are in use, and 428 of these are metered. The average daily consumption is 942,516 gallons, or 112 gallons per capita of the total population of the village and 149 gallons of the population actually supplied. This large consumption can be explained only by the extravagant use of water in public fountains, watering troughs, and for flushing sewers, and by the habit of allowing the water to run constantly when it is muddy.

Crestline.—The public water-supply system of Crestline (population, 3,282) was put in by the village in 1871, and had cost to July, 1898, $225,000. The supply was first obtained from springs 4½ miles east of the village, but these failed to furnish enough water, and in 1895 three wells were drilled to help out. Two reservoirs have been built near the springs to impound the flow from them and from a small area around them. This tributary area is about 3½ square miles in extent, but some of the drainage is diverted from the reservoirs by ditches. One reservoir was built in 1871 and a second in 1888, both having been formed by throwing up earthen embankments across and around the valley of a small run. Only the first reservoir, with a capacity of 1,295,000 gallons, is in use, the second still being very foul from vegetable growths of various kinds, due, no doubt, to an improper stripping of the flooded area. The first reservoir is full of growths also, but here they are due to the exposure of ground water in an unlined open basin. These growths nearly fill the reservoir to the top of the water, and below the living part is a foot or more of thick black mud, formed by the decomposition of moss. The authorities are trying to have this reservoir cleaned, paved, and covered over, so as to do away with the only objectionable feature of the supply.

The wells are 6 inches in diameter and 140, 170, and 180 feet deep. The water flows from them at a rate of 150,000 gallons a day, and it is necessary to pump forty-eight hours a week to keep up the supply. This pumping is done by a 750,000-gallon duplex Worthington pump, installed in 1896. The wells flow and are pumped to the first reservoir, which is 127 feet above the streets of Crestline, and supplies the mains by gravity through an 8-inch wooden-pipe line 4½ miles long. This pipe furnishes an inadequate supply, especially in case of fire, and it is intended to put in a 16-inch cast-iron line. The water seems to reach the consumers clear and tasteless, but it must sooner or later be affected by the bad condition of the reservoir.

There are 10 miles of mains laid, giving about 90 per cent of the people access to the water. There are 445 services connected, of which 400 are in use, and 68 of these are metered. The average daily consumption is 200,000 gallons, or 61 per inhabitant, and 70 per consumer.

DISCUSSION OF WATER SUPPLIES.

From the mass of details contained in the above descriptions of the public water supplies there can be gathered a few interesting facts.

OWNERSHIP.

Of the 104 waterworks 82 are owned and controlled by the city or village, 21 by private corporations, and 1 is under joint private and municipal ownership and direction. Five of the public water supplies are of such poor quality that unless steps are taken for their betterment by the corporations in control the citizens of the towns interested will probably endeavor to purchase the plants and franchises of the companies and procure a good supply under their own management. Even a carelessly conducted municipal plant will usually give better service than most of those under private direction. Under municipal ownership it is much easier to make the changes necessary to the improvement of the quality of the water, though some private companies are far-sighted enough to see that it pays to furnish a potable water. Also, as a rule, the water rates are higher under private than under municipal ownership.

Following are brief outlines of the franchises and contracts under which the private companies operate:

Ada.—The waterworks were put in by the Ada Water, Heat and Light Company, operating under a franchise granted in 1902 and running for twenty-five years. The city may purchase the plant at the end of fifteen years if it so desires. The contract does not specify the kind of water to be furnished.

Blanchester.—The first waterworks were put in by the village, but the supply of water from wells failed almost immediately and the village was without water for nearly two years, as there were no more funds available to seek a new supply.

In order to get fire protection a private company was formed and a franchise secured from the village, according to which the company agreed to furnish a supply of water for the pumps, in return for which they were to collect all the private water rents. This company, the Blanchester Water and Light Company, put in the impounding reservoir at a cost of $6,000. The franchise is to run for fifteen years from 1899, and the village can purchase at the expiration of the franchise only. The water rates are fixed by the franchise, which also specifies that the water shall be taken from the reservoir.

Bucyrus.—The waterworks were put in by the Bucyrus Water Company, operating under a franchise granted in 1883, but it now operates under a new one, granted March 11, 1895, to run fourteen years. Nothing is stated concerning the method by which the city may secure control of the plant before the expiration of the franchise. The contract specifies that the "running" water of the Sandusky River shall

be impounded in a reservoir of 60,000,000 gallons capacity, and "filtered" before it is admitted to the mains. Until this year the water was passed through two pressure filters without previous impounding, but now the supply will be the impounded waters of a small stream tributary to Sandusky River. The water will not be filtered unless filtration is required by the State board of health.

Canal Fulton.—Waterworks were installed in 1902 under a franchise granted March 3 of that year, to run for twenty years. Nothing is stated concerning the method by which the city may secure control of the plant before the expiration of the franchise, and the water company is the sole arbiter as to the kind of water to be furnished and the price to be charged for it.

Defiance.—Waterworks were put in by the Defiance Water Company, operating under a franchise granted in 1887, to run for thirty years. The city may purchase the plant at the end of ten years, or at the end of each five-year period thereafter. The rates are subject to regulation by the city. The franchise provides that "the water shall be taken from such point as is free from all sewage contamination, and shall at all times be a good, wholesome water, fit for all purposes of domestic and manufacturing use." The sewage of a city of 50,000 inhabitants enters Maumee River 54 miles above the point where the water is taken.

Dennison.—Waterworks were put in by the Dennison Water Supply Company, operating under a franchise granted September 5, 1887, to run for twenty years. The village may purchase the plant complete at any time after fifteen years. Rates are fixed by franchise. The franchise provides that the water shall be taken from filter galleries and wells adjacent to Big Stillwater Creek, and that there shall be a direct connection with the creek, to be used in case of emergency. The water was used directly from the creek until January, 1900, at which time mechanical filters were put into operation.

Uhrichsville.—The same company that furnishes Dennison with water supplies Uhrichsville. Here it works under a twenty-year franchise, granted March 6, 1887. The village may purchase at the end of fifteen years such part of the plant as is within the corporate limits of Uhrichsville. The rates are fixed by the franchise, which also provides that the company furnish a good, wholesome water, suitable for domestic use, to be taken from Big Stillwater Creek, or the Tuscarawas River, through wells or galleries, with an auxiliary pipe to the creek or river, for use in cases of emergency. The same water has been furnished that is supplied to Dennison, as noted above.

Massillon.—The first waterworks were put in by a private company in 1854, but the present works are operated by the Massillon Water Supply Company, working under a franchise granted June 7, 1886, and running for twenty years. The city might have purchased the works

at the end of ten or fifteen years. The rates are subject to regulation by the city. The franchise states that the water is to be taken from Sippo Lake, led to an impounding reservoir, filtered, and pumped to a standpipe. A well water is now furnished for domestic use.

Harrison.—A franchise was granted in 1891, to run twenty years, to the Harrison Water and Light Company. It contains no provision for purchase by the city, and the kind of water to be supplied is not stated.

Newark.—The waterworks were put in by the Newark Water Company, working under a franchise granted May 4, 1885, and extending for twenty years. The city may purchase at any time after ten years. Rates are fixed by the franchise. "The water shall be obtained from Berkham Spring, or such other source as may be approved by city council. It shall be clear water, suitable for culinary and drinking purposes, and not detrimental to the health of those who use it." Well water, supplemented by surface water, is now used.

New Comerstown.—The waterworks were put in in 1902 under a franchise granted to the New Comerstown Water Company on July 22, 1901, the franchise to run for twenty years.

The company is to provide "a sufficient supply of good, wholesome water, suitable for domestic, manufacturing, and fire purposes, the same to be obtained from open or driven wells, as long as a sufficient quantity can be obtained."

The water rates are fixed in the franchise, and the city can purchase the plant at the end of ten years, and every five years thereafter, from the date of the passage of the ordinance.

New Philadelphia.—Waterworks were put in by the New Philadelphia Water Company, working under a franchise granted June 3, 1886, to extend for fifteen years. The village may purchase the plant at the end of ten years. The rates are required to be as low as the average for Ohio companies. The franchise requires that the water shall be taken from filtering galleries or wells adjacent to Tuscarawas River.

Piedmont.—As this village is not incorporated, the Piedmont Water Company secured in 1894 a 99-year lease on the springs and the right of way. The lease contains no requirements concerning purchasing rates or kind of water to be supplied.

Oakwood.—The waterworks were put in in 1902 under a franchise granted March 27, 1902, to the Oakwood Water and Light Company, the franchise to run for twenty-five years.

The company is to furnish a "sufficient supply of good water for domestic purposes." This water is obtained from wells.

There is no purchasing clause included in the franchise, and the rates are fixed by the company.

Scio.—The plant was put in by the Scio Water Supply Company, operating under a franchise granted February 18, 1899, to extend for

ninety-nine years. The village may purchase the plant at the end of each five-year period after the completion of the works. The rates are required to be the same as those at Bridgeport, Ohio. The franchise specifies that the water shall be the best obtainable from driven wells.

Shelby.—Waterworks were put in by the Shelby Water Company, operating under a franchise granted February 5, 1896, and extending for twenty years. The village may purchase at the end of ten or fifteen years. The rates are fixed by the franchise, which also requires that the water shall be a good, potable water, free from all obnoxious and injurious elements, and that it be procured from such suitable place as will guarantee its purity, and that the source shall be protected from future encroachments and pollution. Surface water shall not be used.

Tiffin.—Waterworks were put in by the Tiffin Waterworks, operating under a franchise granted December 1, 1879, to run for twenty-five years. The franchise does not regulate the rates or provide for purchase by the city. It states that the water can be taken from such points as best subserve the object and purposes of the company.

Trotwood.—Waterworks were put in by a stock company formed of citizens of the village. This company has no franchise, as the village is not incorporated and none could be granted. The plant was put in by sufferance alone. The mains are in the county and township roads, the verbal permit of the commissioners and trustees having been obtained.

Upper Sandusky.—Waterworks were put in by the Upper Sandusky Waterworks Company, operating under a twenty-year franchise granted April 23, 1889. The village may purchase the plant at the end of ten or fifteen years. Rates are not regulated in any way. The franchise states that the water shall be taken from wells sunk in the gravel banks of Sandusky River or it shall be "river water dispossessed of earthy matter by settling pools." The well water shall be carefully protected from pollution and shall be increased from time to time as the consumption warrants. A separate suction pipe may be run to the river, to be used in cases of great emergency only. The river water is used practically direct nearly all the time.

Urbana.—The franchise of the Urbana Waterworks Company expired in 1899, and no new one has been secured since that time. There has been considerable friction between the city and the company, and the terms of the settlement are uncertain. The company still supplies water for the citizens and collects the water rent, and this condition may hold for some time.

Wadsworth.—Waterworks were put in by the Wadsworth Waterworks Company, operating under an unlimited franchise granted September 20, 1881. Under the first grant the village could purchase at the end of ten years, but this grant was amended July 6, 1892, and

the right of purchase given up. It is practically a perpetual franchise. Rates are fixed by the franchise.

Waynesburg.—In 1857 the Waynesburg Water Company was granted the use of the village streets for a free water trough and one fire plug. The grant states nothing regarding the purchasing of works, regulation of rates, or kind of water to be supplied.

Xenia.—This city is supplied with water by the Xenia Water Company, operating under a twenty years' franchise granted July 19, 1886. The city may purchase the plant at any time after the expiration of the first ten years. Rates are fixed by the franchise. The water is to be taken from wells or filtering galleries near Little Miami River or Cæsars Creek, with direct connection to either stream for fire protection only. As a matter of fact neither stream is used, but the water is procured from another source.

Lack of potable water.—It is unfortunate that in many cases a nonpotable water is supplied, even when the franchise specifically requires a water that is of good quality, or that is derived from a source that would furnish a potable water. In so many communities the desirability of a good water has not been so realized by those in authority as to impel them to take the steps necessary to enforce the provisions of the waterworks franchises. When the present franchises expire, if it is not possible to purchase the works outright, it would be well for the interested towns to see that the new franchises contain a very definite clause in reference to the kind of water to be supplied.

DATE OF INSTALLATION OF WATERWORKS.

The following table shows the decades in which water supplies were introduced in the various towns:

Basin.	1840–1850.	1850–1860.	1860–1870.	1870–1880.	1880–1890.	1890–1900.	1900–1903.
Sandusky				2	3	3	
Maumee				1	3	11	5
Miamis			1	4	6	16	7
Muskingum	1	2	1	2	7	18	6
Total	1	2	2	9	19	48	18

This shows that 66⅔ per cent of the plants have been put in since 1890 and that about 86 per cent have been put in since 1880, confirming the general fact that public sentiment requiring better water than that furnished by a private well is of very recent growth. In many cases, however, a demand for fire protection has been almost the sole reason for the introduction of a public supply. The number of old plants in the Muskingum drainage basin accords very well with the fact that it is a portion of the State that was developed at a very early period.

METHOD OF SUPPLYING WATER.

A study of the methods used in the various drainage basins to supply the water shows the general nature of the country as to levelness almost as well as a topographic map. In the Sandusky basin there is 1 gravity supply, 2 direct pumping, and 5 pumping to standpipes or elevated tanks. The Maumee basin, with its broad, level plains, has not developed a single supply that does not involve pumping; 8 direct and 12 by pumping to standpipes or elevated tanks. The Miami basins also have no gravity supplies, but in 13 cases the pressure is obtained by direct pumping, in 16 cases by pumping to standpipes and tanks, and in 3 cases only by pumping to reservoirs. On the other hand, the rather hilly Muskingum district has 4 gravity supplies, 4 in which the water is furnished by gravity and pumping to reservoirs, 1 by gravity and pumping to a standpipe, 11 by pumping to reservoirs, 1 by pumping to reservoirs and a standpipe, 13 by pumping to standpipes, and only 4 supplied by direct pumping.

The safety and convenience of a gravity supply from a reservoir, or a supply from a reservoir filled by pumping, are generally recognized, and wherever the topography will permit these methods are used.

The 5 gravity supplies of the basins are small and unimportant—in fact, all the surface supplies that depend upon impounding reservoirs are not large. In general, Ohio cities are content to seek a ground water, if they are not near a large river whose water may be used directly, rather than to go to the expense of impounding the runoff of the smaller streams and protecting the basin from pollution. The excessive turbidity of these small streams is also an important factor which works against the use of this water, and it is turbidity that makes all the surface supplies objectionable for a good part of the year.

The present industrial supply for Massillon is obtained, as was the old supply, from an impounding reservoir which has an estimated capacity of 30,000,000 gallons.

Bellefontaine has an impounding reservoir of about 2,000,000 gallons capacity, Blanchester one of 5,000,000 gallons, Hamilton one of 7,000,000 gallons; Wooster has one of about 8,000,000 gallons, and Crestline two of about 1,000,000 gallons capacity each. These, together with 21 storage and distributing basins, constitute the reservoirs of the four drainage areas. Uhrichsville and Dennison, Lebanon, New Philadelphia, Springfield, and Zanesville have good storage reservoirs of large capacity for this class of work.

MILES OF MAINS AND POPULATION ACCESSIBLE.

In the 104 towns there are 1,601.6 miles of mains, or an average of 15.4 miles for each place. On an average, 85 per cent of the total population of each town has access to the water; the various percent-

ages range from 50 to 95. Considering the towns as a whole, 601,480 people have access to the public water supplies; this is 78 per cent of the total population (771,128) of the towns supplied. The remaining 169,648 people, or 22 per cent, must still depend upon private wells until such time as the officials in charge shall see fit to extend the mains.

POPULATION ACTUALLY SUPPLIED.

The number of people actually using the water has been estimated from the number of services in use at 393,275 for the four drainage areas, divided as follows: Sandusky, 18,280; Maumee, 102,695; Miamis, 138,740; Muskingum, 132,560. Of the total population the above number who use the water constitute only 51 per cent, and but 66 per cent of the population accessible to the water. Thus, while 22 per cent of the population of towns with public water supplies do not use the water from lack of access, 34 per cent of those who can use it do not do so, some of these from lack of means, but many from false ideas as to the purity and potability of the water obtained from private wells. Unfortunately, it is necessary to add that in too many cases the public supply is of no better quality than the water from the private wells.

USE OF METERS.

Of the 80,467 services in use, only 12,875, or 16 per cent, are metered. Leaving out Toledo, with its 13,961 services and 7,200 meters, only 8.5 per cent of the services in use are metered. Of the 104 public supplies, 25 have not a single meter in use and 27 others have 10 or fewer. Shreve has all of its 85 services metered, West Alexandria all of its 131, and Cambridge 130 of its 140 services. Attica, Bucyrus, Dayton, Eaton, Greenville, Hamilton, Norwood, Toledo, and Wooster have more than one-fourth of their services metered. These towns, with Shreve, West Alexandria, and Cambridge, comprise all the towns where meters are used to any extent for domestic services. In the remaining places meters are used, as a rule, only for manufacturing establishments, livery stables, etc.

PUMPING CAPACITY.

It might be well to call attention to the large reserve pumping capacity at hand, as so many of the towns are supplied directly or by small reservoirs and standpipes. This excess is found to vary from 2 to 23 times the average daily consumption. The high figures occur in new plants, where the consumption is not yet great and a large allowance has been made for future growth. There is only one supply in the four basins where water power is used in public waterworks. Tiffin utilizes the water of the Sandusky River to run a 1,500,000-gallon Holly pump, which usually supplies the pressure during the night only.

AVERAGE DAILY CONSUMPTION.

In 84 of the 104 supplies it was possible to estimate the average daily consumption. In a number of cases accurate daily records of the pumps are kept and the results have some weight, but in others merely the crudest estimate can be made.

Considering the 84 towns in which the consumption could be estimated, the average daily consumption per capita of the total population is 70 gallons, and per consumer, or per capita of the population actually supplied, it is 125 gallons. In individual towns these figures run from 150 gallons as the average daily consumption per consumer and 17 gallons per capita of the total population to 106 and 100 gallons, respectively. The minimum average daily consumption is found in the small places, especially in those where there are no sewers. There are not enough metered supplies to show the effect of meters on the consumption. The maximum normal consumption is found in those towns where the supply is of poor quality, especially if it is turbid; for example: Cambridge, 442 gallons; Dennison and Uhrichsville, 442; Waynesville, 218; Wooster, 210; Zanesville, 259; Defiance, 300; Bucyrus, 280, and Fremont, 350. Shelby had an average daily consumption per consumer of 330 gallons, but this was due to the construction and flushing of a complete new sewer system.

The variation of the consumption throughout the year was carefully determined from the monthly pumping records of ten supplies in the Muskingum basin—Ashland, Canal Dover, Canton, Carrollton, Dennison and Uhrichsville, Massillon, Mount Vernon, Newark, Orrville, and Zanesville—in all, about 450 monthly records. With the average monthly consumption equaling 100, the following gives the variation:

Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
104.8	99.0	95.2	88.7	91.3	102.9	102.6	113.4	104.1	103.9	95.9	98.0

The increase in the consumption in the cold months is due to the fact that hydrants are permitted to run to prevent freezing, and the increase in the summer is due to the sprinkling and general free use of water. The increase in consumption from year to year is well shown in the following 18-year record for Mount Vernon:

Yearly variation in consumption at Mount Vernon.

Year.	Average daily consumption.	Estimated population.	Estimated population using water.	Average daily consumption per capita.	Average daily consumption per consumer.
1884	213,774	5,540	3,270	39	65
1885	280,346	5,620	3,300	50	85
1886	281,271	5,680	3,360	50	84
1887	346,622	5,780	3,420	60	101
1888	361,247	5,860	3,480	62	104
1889	339,676	5,940	3,780	57	90
1890	354,672	6,027	3,960	59	90
1891	399,386	6,088	4,200	66	95
1892	500,091	6,149	4,550	81	110
1893	598,066	6,209	4,800	96	125
1894	636,662	6,270	5,100	101	125
1895	732,932	6,331	5,400	116	136
1896	688,311	6,392	5,580	108	123
1897	794,898	6,452	5,820	123	137
1898	814,567	6,513	6,000	125	136
1899	858,510	6,574	6,150	130	140
1900	926,157	6,663	6,335	139	146
1901	1,000,000	6,694	6,500	149	154

Where this increase will stop is a problem hard to solve. That a great deal of the increase is due to waste is undoubted, but it is difficult to determine just how much is due to legitimate increase of use. In a few places, by the general introduction of meters, and by carefully watching for leaks, the consumption per capita has been greatly decreased and then held to a low rate of increase. As the population increases it becomes difficult to supply an abundance of potable water, and no doubt it will soon be necessary in many places to take stringent steps to check waste.

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DEPARTMENT OF THE INTERIOR

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

THE PASSAIC FLOOD OF 1903

BY

MARSHALL ORA LEIGHTON

WASHINGTON

GOVERNMENT PRINTING OFFICE

1904

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
HYDROGRAPHIC BRANCH,
Washington, D. C., December 4, 1903.

SIR: I have the honor to transmit herewith a manuscript entitled, "Passaic Flood of 1903," prepared by Marshall Ora Leighton, and to request that it be published as one of the series of Water-Supply and Irrigation Papers.

This paper is a continuation of Water-Supply and Irrigation Paper No. 88, by George B. Hollister and Mr. Leighton, and describes the flood of October, 1903, which was higher and far more disastrous than the flood of 1902. The occurrence of two great floods in the same basin during so short a period makes the subject worthy of attention, especially as the district is, from a manufacturing and commercial standpoint, one of the most important along the Atlantic coast.

Very respectfully,

F. H. NEWELL,
Chief Engineer.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

THE PASSAIC FLOOD OF 1903.

By MARSHALL O. LEIGHTON.

INTRODUCTION.

In the following pages is given a brief history of the disastrous flood which occurred in the Passaic River Basin in October, 1903. In the report by George Buell Hollister and the writer, entitled "The Passaic Flood of 1902," and published by the United States Geological Survey as Water-Supply and Irrigation Paper No. 88, are discussed the principal physiographic features of the drainage basin and their general relations to the stream flow. This report will not repeat this information, and the discussion will be confined to the flood itself. References to local features will be made without explanation, the presumption being that this publication shall accompany the earlier one and be, as it is, a continuation of it. In the present report more attention is given to an estimate of damages than in the earlier work, and remedies by which devastation may be avoided are briefly considered.

Passaic River overflowed its banks on October 8, 1903, and remained in flood until October 19. Between these dates there occurred the greatest and most destructive flood ever known along this stream. Ordinarily the channel of the lower Passaic at full bank carries about 12,000 cubic feet of water per second, but at the height of this flood it carried about 35,700 cubic feet per second.

The flood period for the entire stream can not be exactly stated, as the overflow did not occur at the same time in different parts of the basin. For example, the gage-height records at Dundee dam show that the flood began to rise on October 8 at 6.30 a. m., and reached a maximum of 9½ inches over the dam crest at 9 p. m. on October 10. Similarly, on Beattie's dam at Little Falls the flood began to rise at midnight on October 7, and reached its maximum at 2 p. m. on October 10, or about thirty-eight hours after the initial rise, the height of the water being 1.29 inches over the crest of the dam.

The flood rose on the highland tributaries as follows: On Ramapo River the flood crest passed Hillborn at about 10 a. m. on October 9

and reached Pompton, at the mouth of the river, shortly after noon of the same day.

The highest reading recorded on the Geological Survey gage at the feeder of Morris Canal, in Pompton Plains, was 14.3 feet, at about 6 o'clock on the morning of October 10. As this gage is read only once daily it is probable that this reading does not represent the height of the flood crest. Evidence shows that it passed this point on the previous day. Records of the Newark water department show that the flood on Pequanac River began to rise at Macopin dam on October 8 at noon, and rose rapidly to the maximum of 6,000 cubic feet per second at 4 p. m. on October 10.

No records are available with reference to the rise of flood on Wanaque River.

Observations made on Pompton Plains on the morning of the 11th show that Pompton River was well within its banks at that time; therefore the Ramapo, Wanaque, and Pequanac must have discharged their flood waters some time previous to this hour. The fact is important when considered in connection with the height of water in the main stream at that period. This observation was made only eighteen hours after the maximum height over Beattie's dam at Little Falls, and twelve hours after the flood crest passed Dundee dam. The conditions here outlined illustrate the rapidity with which flood waters are discharged from the Pompton drainage area, and the deterring effect of Great Piece Meadows upon the flood.

The rise of the flood on Rockaway River at Old Boonton was almost coincident with that on Pequanac River at Macopin dam. The maximum flow occurred fourteen hours later than the maximum on the Ramapo at Pompton.

The flood crest did not reach Chatham on upper Passaic River until the morning of October 11, or about twenty-four hours later than the flood heights in Pompton and Rockaway rivers, and about twelve hours later than the maximum over Dundee dam.

Adequate reasons for these differences in flood periods between neighboring points are abundant. They are apparent after a review of the physiographic conditions described in Water-Supply Paper No. 88.

The flood of 1903 was the immediate result of an enormous rainfall, and not, as is often the case in north temperate latitudes, the combined effect of rainfall and the rapid melting of accumulated snows. The records of weather-observation stations in northern New Jersey and New York fail to show, throughout their entire observation periods, as great an amount of precipitation in so short a period. The storm which was the immediate cause of the flood occurred principally between October 8 and 11. During that interval rain fell to an average depth of 11.74 inches over the Passaic Basin.

The Passaic Basin is fairly well supplied with storage facilities, which, under ordinary circumstances, would temper the severity of floods by holding back a large amount of water. In this case no such effect was produced, as the reservoirs, lakes, and ponds on the drainage area were filled, or practically so, at the beginning of the storm, and there was consequently no available space in which to hold back even an appreciable part of the run-off water. Over some of the dams in the highland region a comparatively small amount of water was being discharged at the beginning of the storm. Therefore, while these storage basins may have had a certain deterring effect upon the rate of flood accumulation, they could not, in the end, assist materially in preventing damages in the lower part of the drainage area.

PRECIPITATION.

The precipitation records for June, July, August, and September are given below:

Precipitation, in inches, in Passaic Valley and vicinity, June to September, 1903.

	June.		July.		August.		September.	
	Normal.	Observed.	Normal.	Observed.	Normal.	Observed.	Normal.	Observed.
Highland region:								
Dover	3.29	15.02	5.54	5.47	5.08	9.04	4.02	3.39
Chester	3.48	12.80	6.42	7.59	5.16	9.35	4.60	.....
Charlotteburg	3.52	9.45	5.54	3.97	4.98	7.78	4.80	3.29
Ringwood	.....	10.13	.....	3.08	.....	6.17	.....	3.06
Red Sandstone plain:								
Paterson	4.31	11.17	5.32	5.40	4.31	10.89	4.86	2.88
Hanover	3.32	.....	5.23	5.40	5.20	9.40	4.52	.....
River Vale	3.17	10.62	4.87	3.41	4.17	.....	3.61	2.90
Essex Fells	3.08	.....	7.03	.....	5.95	.....	3.67	1.80
Newark	3.60	11.51	4.48	4.27	4.75	14.54	3.83	4.56
South Orange	3.57	9.28	5.43	4.22	5.05	13.75	4.04	3.80
New York City	3.13	7.42	4.26	3.23	4.70	5.96	3.72	2.60
Plainfield	3.62	10.14	5.86	4.70	4.37	6.87	4.42	7.10
Elizabeth	3.68	8.76	5.74	4.31	4.26	7.15	4.14	4.38

An examination of the above table shows that throughout the summer of 1903 the precipitation was considerably above normal. The records for June and August indicate extremely wet months, and the July figures are slightly above while the September figures are somewhat below normal. The important fact shown by this table is

that disastrous floods may occur after long periods of abundant rains. It has been observed that heavy precipitation may be expected after protracted periods of drought. Such a belief is not altogether fanciful. In the northeastern part of this country the total amount of precipitation is approximately uniform from year to year. The variations, comparatively speaking, are not very wide, and we are therefore led to expect that there are in operation influences which serve to compensate for excesses or deficiencies in our annual rainfall. Therefore after the abundant precipitation of the summer of 1903, an observer might have had some measure of justification in predicting a normally or abnormally dry fall. In view of the actual events the fact must be emphasized that in adopting measures to prevent floods the margin of safety must be extremely wide. The extraordinary rainfall of those three October days can not with assurance be accepted as the maximum.

Precipitation, in inches, in Passaic Valley and vicinity, October 7 to 11, 1903.

Station.	From—		To—		Amount.
	Day.	Hour.	Day.	Hour.	
Highland region:					
Dover	7	.....	11	9 p. m	10.13
Little Falls	7	4 a. m	11	7 a. m	14.13
Charlotteburg	7	.....	10	.....	12.67
Ringwood	8	11 a. m	9	8 p. m	10.63
Red Sandstone plain:					
Paterson	7	5 a. m	9	3.45 p. m	15.04
River Vale	8	8 a. m	11	6 p. m	12.55
Essex Fells	8	.....	9	4 p. m	10.66
Newark	8	8.30 a. m	11	5 a. m	12.09
South Orange	8	6 a. m	10	Night	10.48

The extremely rapid rate of precipitation during the crucial part of the storm is shown by the recording gages placed at observation stations in Newark and New York City.

Hourly records of precipitation at New York observation station, October 8 and 9, 1903.

	Inches.		Inches.
Oct. 8, 9 to 10 a. m	0.08	Oct. 9, 1 to 2 a. m	0.25
10 to 11 a. m	.02	2 to 3 a. m	.75
11 to 12 m	.32	3 to 4 a. m	.34
12 m. to 1 p. m	.10	4 to 5 a. m	.46
1 to 2 p. m	.05	5 to 6 a. m	.41
2 to 3 p. m	.06	6 to 7 a. m	.29
3 to 4 p. m	.34	7 to 8 a. m	.51
4 to 5 p. m	.01	8 to 9 a. m	1.38
5 to 6 p. m	.10	9 to 10 a. m	1.04
6 to 7 p. m	.02	10 to 11 a. m	.08
7 to 8 p. m	.93	11 to 12 m	.23
8 to 9 p. m	.32	12 m. to 1 p. m	.24
9 to 10 p. m	.24	1 to 2 p. m	.31
10 to 11 p. m	.27	2 to 3 p. m	.32
11 to 12 p. m	.26	3 to 4 p. m	.01
9, 12 to 1 a. m	.30	Total	6.92

Hourly record of precipitation at Newark observation station, October 8–11, 1903.

	Inches.		Inches.
Oct. 8, 8.25 to 9 a. m	0.05	Oct. 9, 7 to 8 a. m	0.29
9 to 10 a. m	.04	8 to 9 a. m	.69
10 to 11 a. m	.00	9 to 10 a. m	.69
11 to 12 m	.00	10 to 11 a. m	.39
12 m. to 1 p. m	.14	11 to 12 m	.20
1 to 2 p. m	.72	12 m. to 1 p. m	.39
2 to 3 p. m	.49	1 to 2 p. m	.28
3 to 4 p. m	.11	2 to 3 p. m	.34
4 to 5 p. m	1.05	3 to 3.25 p. m	.13
5 to 6 p. m	.45	11.50 to 11.55 p. m	.01
6 to 7 p. m	1.20	10, 3 to 4 a. m	.02
7 to 8 p. m	.60	7 to 8 p. m	.07
8 to 9 p. m	.24	8 to 9 p. m	.09
9 to 10 p. m	.24	9 to 10 p. m	.02
10 to 11 p. m	.13	10 to 11 p. m	.04
11 to 12 p. m	.17	11 to 12 p. m	.04
9, 12 to 1 a. m	.29	11, 12 to 1 a. m	.06
1 to 2 a. m	.33	1 to 2 a. m	.09
2 to 3 a. m	.62	2 to 3 a. m	.03
3 to 4 a. m	.29	3 to 4 a. m	.05
4 to 5 a. m	.35	4 to 5 a. m	.01
5 to 6 a. m	.26	Total	11.83
6 to 7 a. m	.13		

From the above tables it may be seen that the maximum rate of precipitation per hour was 1.38 inches at New York and 1.2 inches at Newark. Comparison of the tables on pages 11 and 12 gives an excellent idea of the intensity of the storm. The amount of water falling in a single storm is nearly equal to the total for June, a month of unusual precipitation.

The average of the total amounts of precipitation recorded at the various stations in the Passaic area is 11.74 inches. These totals are fairly uniform, none of them varying widely from the average. Therefore the figure 11.74 represents a conservative mean for a calculation of total amount of water over the drainage area. Assuming this as the correct depth, the amount of water which fell on each square mile of the Passaic drainage area during the storm was 27,273,000 cubic feet, or for the whole Passaic drainage area over 27,000,000,000 cubic feet, weighing about 852,000,000 tons. This amount of water would, if properly stored, fill a lake with twenty times the capacity of Greenwood Lake, would cover Central Park in New York City, which has an area of about 1.5 square miles, to a height of 645 feet, and, at the present rate of water consumption in the city of Newark, N. J., would supply the city with water for twenty years.

DESCENT OF FLOOD.

HIGHLAND TRIBUTARIES AND CENTRAL BASIN.

A description of the descent of flood waters from the highland tributaries into the Central Basin has been given in Water-Supply Paper No. 88. It has been shown that the lands of the Central Basin are covered even in ordinary freshets, and that in the event of a great flood the waters merely rise higher, being, for the greater extent, almost quiescent, and beyond the flooding of houses and barns and the destruction of crops, little damage is done. In other words, the flood along this portion is not torrential in character.

During the flood of 1903 the water fell so quickly all over this basin, and was collected so rapidly by the small tributaries, that a lake was formed at once which served as a cushion against which the raging torrent of the highland tributaries spent itself without doing extraordinary damage in that immediate region. Bridges which might have been lost in a smaller flood like that of 1902 were actually standing in slack water by the time the mountain torrents appeared in force. These streams caused much destruction higher up in the mountains, but in the Central Basin their energy became potential—a gathering of forces to be loosed upon the lower valley. A discussion of the effects of this will be taken up under the heading "Damages."

In Water-Supply Paper No. 88 is given the proportion of flood waters

contributed to the Central Basin by each of the tributaries. These figures were computed from the results of gagings maintained for a period sufficient to afford this information within a reasonable approximation. In the case of the storm which resulted in the flood of 1903 it is probable that data referred to can not be safely applied.

The flood of 1902 was the result of abundant rains following upon and melting a heavy snow. Weather Bureau records show that neither the depth of the snow nor the amount of subsequent rainfall was uniform, or even approximately so, over the Passaic drainage area. Indeed, so marked was the variation that it was believed that the mean rainfall for all the observation stations on the basin did not bear sufficient relation to observed run-off to allow of any reliable deductions. In the case of the October storm, however, the distribution of rainfall was more nearly uniform, and the run-off from the highland tributaries into the Central Basin must have been proportionately different in amount from that indicated in the upland tributary tables in the report of the previous flood. The data given for the 1902 flood can not, therefore, in the case of the highland tributaries, be applied to the conditions which obtained in the flood of 1903.

FLOOD AT MACOPIN DAM.

Mr. Morris R. Sherrerd, engineer of the Newark city water board, has furnished flow computations over Macopin intake dam, which is the head of the Newark pipe line. As about 73 per cent of the Pequanac drainage area lies above this intake, the table on page 16 shows roughly an equivalent percentage of the flow contributed by Pequanac River to the Central Basin of the Passaic. In consulting this table it should be borne in mind that the entire run-off of the drainage area above Macopin is about 25,000,000 gallons per day more than the amounts presented in this table. All reservoirs and ponds connected with the conservancy system of the Newark water supply were filled except that at Oakridge, which was about 1.5 feet below the crest of the spillway.

Flow of Pequanac River over Macopin dam, October 7–24, 1903.

[From Newark water department.]

	Cubic feet.		Cubic feet.
Oct. 8, 6 a. m. to 12 m.....	240,600	Oct. 14, 12 to 6 a. m.....	9,626,000
12 m. to 4 p. m.....	347,600	6 a. m. to 12 m.....	8,690,000
4 to 6 p. m.....	842,200	12 m. to 6 p. m.....	8,022,000
8–9, 6 p. m. to 6 a. m..	40,110,000	6 to 12 p. m.....	7,353,000
9, 6 a. m. to 12 m.....	51,870,000	15, 12 to 6 a. m.....	6,952,000
12 m. to 1 p. m.....	15,100,000	6 a. m. to 6 p. m....	12,700,000
1 to 5 p. m.....	62,430,000	15–16, 6 p. m. to 6 a. m.	10,965,000
5 to 10 p. m.....	89,040,000	16, 6 a. m. to 6 p. m....	10,025,000
10 to 11 p. m.....	19,520,000	16–17, 6 p. m. to 6 a. m.	9,091,000
9–10, 10 p. m. to 8 a. m.	201,350,000	17, 6 a. m. to 6 p. m....	8,690,000
10, 8 a. m. to 12 m....	75,670,000	17–18, 6 p. m. to 6 a. m.	9,893,000
12 m. to 6 p. m....	103,650,000	18, 6 a. m. to 6 p. m....	10,565,000
6 to 12 p. m.....	73,530,000	18–19, 6 p. m. to 6 a. m.	8,690,000
11, 12 to 6 a. m.....	56,820,000	19, 6 a. m. to 6 p. m....	6,952,000
6 a. m. to 12 m....	41,440,000	19–20, 6 p. m. to 6 a. m.	6,150,000
12 m. to 6 p. m....	32,755,000	20, 6 a. m. to 6 p. m....	5,882,000
6 to 12 p. m.....	25,665,000	20–21, 6 p. m. to 6 a. m.	5,749,000
12, 12 to 6 a. m.....	23,800,000	21, 6 a. m. to 6 p. m....	5,481,000
6 a. m. to 12 m....	20,725,000	21–22, 6 p. m. to 6 a. m.	5,214,000
12 m. to 6 p. m....	18,450,000	22, 6 a. m. to 6 p. m....	4,144,000
6 to 12 p. m.....	15,105,000	22–23, 6 p. m. to 6 a. m.	3,677,000
13, 12 to 6 a. m.....	13,370,000	23, 6 a. m. to 6 p. m....	3,877,000
6 a. m. to 12 m....	11,890,000	23–24, 6 p. m. to 6 a. m.	5.749,000
12 m. to 6 p. m....	11,230,000	24, 6 a. m. to 6 p. m....	5,615,000
6 to 12 p. m.....	11,230,000		

FLOOD AT BEATTIE'S DAM, LITTLE FALLS.

The flow over Beattie's dam at Little Falls, has been calculated according to coefficients used for the same dam in Water-Supply Paper No. 88. Recorded gage heights show that over the main dam there was a maximum depth of 11.12 feet, which continued from 2 to 8 p. m., on October 10, representing a maximum flow of 31,675 cubic feet per second. (See Pl. I, *A*.) In the following table is set forth the flow of the river over Beattie's dam during the flood, and for purposes of comparison, the figures for the flood period of March, 1902. It should be borne in mind in consulting this table, that in the case of the flood of 1903 exact dates and hours are given, while the figures for the 1902 flood represent flow determinations at six-hour intervals, beginning with the initial rise of that flood.

A. BEATTIE'S DAM, LITTLE FALLS, N. J., IN FLOOD.

B. FLOOD-WATER LINES IN RESIDENCE DISTRICT, PATERSON, N. J.

Flood flow over Beattie's dam during floods of 1902 and 1903.

Date and hour.	1903.	1902.[a]	Date and hour.	1903.	1902.[a]
	Sec.-feet.	*Sec.-feet.*		*Sec.-feet.*	*Sec.-feet.*
Oct. 8. 12 p. m.	1,645	490	Oct. 14. 12 m	11,740	22,650
9. 6 a. m.	4,235	700	6 p. m.	10,975	22,350
12 m	8,560	1,350	12 p. m.	9,820	22,100
6 p. m.	15,755	2,120	15. 6 a. m.	9,180	21,150
12 p. m.	23,927	3,540	12 m	8,330	19,900
10. 6 a. m.	28,370	4,250	6 p. m.	7,700	18,900
12 m	31,305	4,600	12 p. m.	7,005	17,350
6 p. m.	31,675	5,000	16. 6 a. m.	6,695	15,750
12 p. m.	30,770	6,500	12 m	5,920	13,900
11. 6 a. m.	29,840	7,600	6 p. m.	5,620	13,300
12 m	28,950	8,250	12 p. m.	5,360	11,800
6 p. m.	26,960	9,000	17. 6 a. m.	4,855	10,650
12 p. m.	25,530	10,200	Below full bank		8,900
12. 6 a. m.	24,435	11,450	Do		8,500
12 m	22,625	14,700	Do		8,100
6 p. m.	20,810	18,150	Do		8,200
12 p. m.	18,655	20,650	Do		7,000
13. 6 a. m.	17,930	22,200	Do		6,250
12 m	16,190	22,700	Do		5,900
6 p. m.	14,900	23,400	Do		5,300
12 p. m.	13,615	23,300	Do		5,200
14. 6 a. m.	12,340	22,950	Do		4,900

[a] At six-hour intervals.

FLOOD FLOW OVER DUNDEE DAM.

The flood, as indicated by gage heights at Dundee dam, lasted from about 6.30 p. m. October 8 to about midnight October 18. Although the maximum recorded gage height was 19 inches higher than during the flood of 1902, the actual time during which the river was out of its banks was forty-five hours less than at the earlier flood. Examination of fig. 1 shows that the flood of 1903 was decidedly more intense than that of 1902, the maximum height being reached in 1903 in about sixty hours, while in 1902 the maximum was not reached until the expiration of about one hundred and twenty hours.

At Dundee dam the familiar break in the progress of the flood took place about thirty-five hours after the initial rise. It occurred before the time of the maximum gage height at the mouth of Pompton River, and there is nothing to indicate that it was caused, as has been claimed, by slack water from the Pompton flood being forced back into Great Piece Meadows. There is no doubt that a part of the Pompton flood was so diverted, but there was maintained throughout at Little Falls a steady pressure, which constantly increased to maximum. This flood check at Dundee dam was observed in 1902, but it could not be shown to arise from the frequently mentioned phenomena at the mouth of Pompton River. It is important to prove or disprove this hypothesis.

If it were found to be true, it could be advantageously taken into consideration in connection with measures for the prevention of flood damages. As the Pompton had no such effect upon the flood flow at Dundee dam in two consecutive historic floods, the writer is inclined to believe that the idea is entirely erroneous.

Since the flow curves in fig. 1 were drawn it has been found by careful observation that the depressions which occur in the rise of every flood over Dundee dam are probably due to the carrying away of the flashboards which are placed upon the dam crest in times of low water. A review of the gage heights recorded by floods for several years past shows that the break occurs when the height of water over the dam crest reaches from 40 to 60 inches. The flashboards used upon this

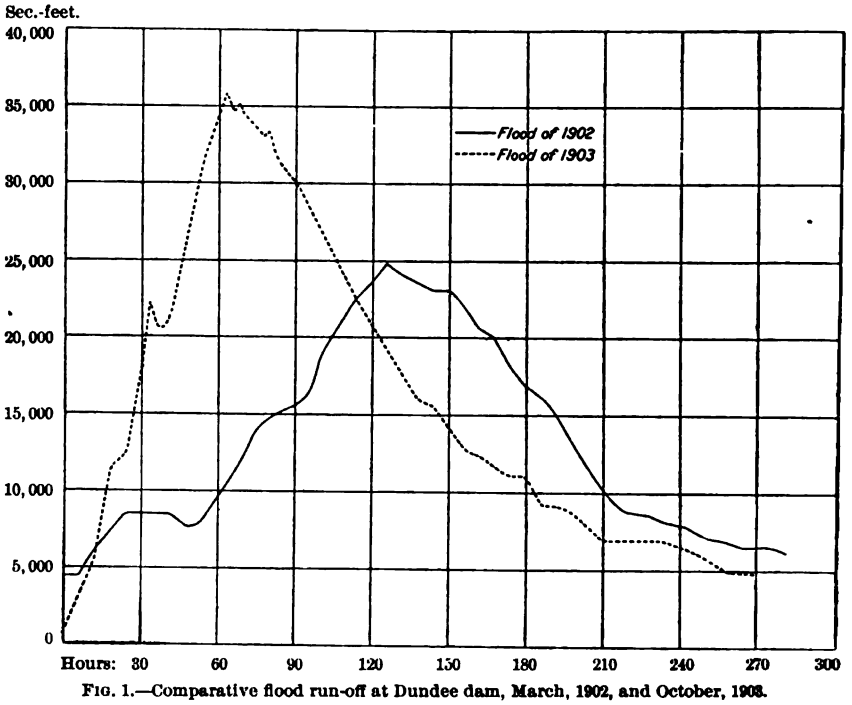


FIG. 1.—Comparative flood run-off at Dundee dam, March, 1902, and October, 1903.

dam are usually 18 inches wide, and as they are supported by iron rods, which are of approximately the same strength and are placed upon the dam by one crew of workmen, it may be safely assumed that they are of approximately equal stability and might be expected to fail almost simultaneously along the length of the dam crest. So sudden a decrease in the effectual height of the dam must lower the water on the dam crest markedly, and as every other probable cause has been eliminated in the case of the recent flood, the explanation of the check in the progress of floods over this dam may be safely accepted as due to carrying away of flashboards. This effect should be apparent in the gage-height records only.

In the flow diagrams (figs. 1 and 2) the effect would not be the same, but the curve would rise more sharply. Similarly, the measurements at the beginning are not correct, as they are calculated according to gage heights measured from the stone crest of the dam. Therefore, a true flood curve at this point would be much flatter at the beginning and rise sharply at a period coincident with the carrying away of the flashboards.

An important difference between the two floods is that the earlier continued longer, but the later one was much higher. The flood of 1902 was caused by the turning of an equivalent of approximately 6 inches of precipitation into the main channel during a period of six days. In the deluge of 1903 there fell 11.74 inches of rain, the greater part of which was precipitated in 36 hours. Thus it is seen that there was in the flood of 1903 a larger rainfall during a much shorter period than in the flood of 1902. Computation shows that the total run-off from the drainage area above Dundee dam during the earlier flood was 13,379,000,000 cubic feet, and that on account of the frozen condition of the ground at that time this amount of water represented practically all of the precipitation. During the flood of 1903 there was a total run-off for the same area of 14,772,000,000 cubic feet, which represents about 66 per cent of the observed precipitation. According to these figures the total amount of run-off in the 1903 flood was only 10 per cent greater than that in 1902, while the actual flood height during the 1903 flood was 27 per cent higher than during the flood of 1902. The above comparison shows, in a striking manner, the effect of the condition of the surface. In the case of the later flood we had, as has been stated in previous pages, an area which had been well watered during the previous summer, and the observed ground-water levels were fairly high. There was, however, sufficient storage capacity in the basin to retain about 34 per cent of the precipitation occurring between October 7 and 11. This water must have been largely absorbed by the earth. The general relations of the floods of 1903 and 1902 can therefore be briefly stated as follows:

General relations of floods of 1903 and 1902.

	Average precipitation.	Duration of precipitation.	Maximum flood flow.	Total run-off.	Run-off.	Duration of flood at Dundee dam.
	Inches.	*Days.*	*Sec.-feet.*	*Cubic feet.*	*Per cent.*	*Hours.*
1902.....	6	6	24,800	13,379,000,000	[a] 100	270
1903.....	11.74	3	35,700	14,772,000,000	66	225

[a] Approximately.

In the following table and fig. 2 are recorded gage heights taken at hourly intervals during the crucial part of the flood and the amount of water expressed in cubic feet per second flowing over the crest of the dam at each gage height.

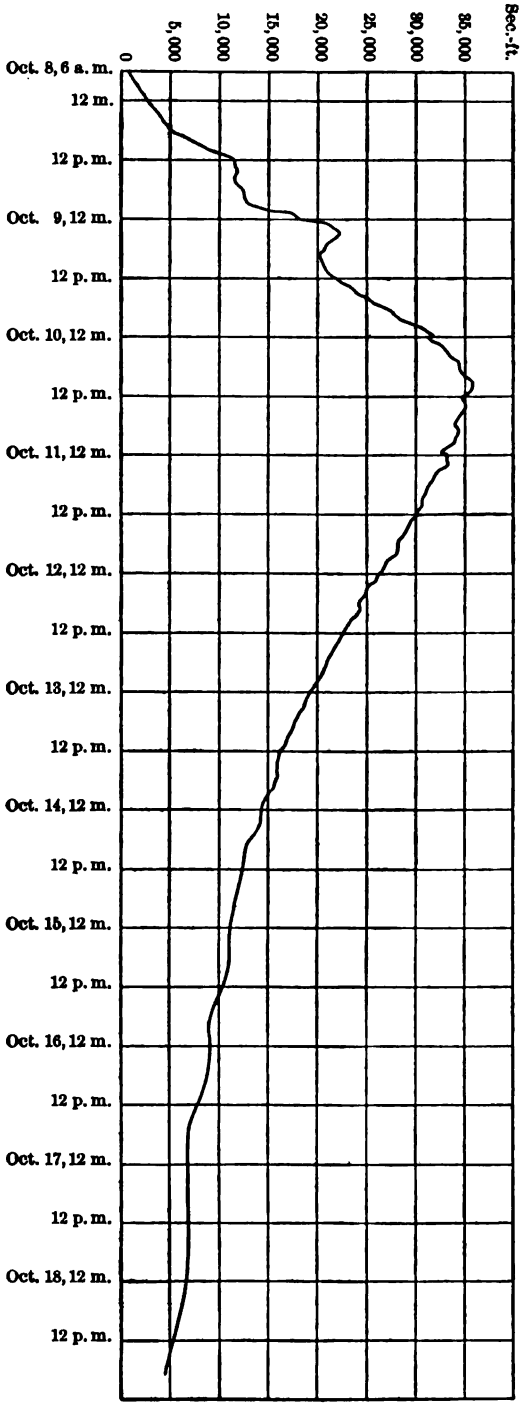


FIG. 2.—Diagram of flood flow at Dundee dam, flood of 1903.

Flow of Passaic River at Dundee dam, 1903.

Date and hour.	Gage.	Flow.	Date and hour.	Gage.	Flow.
	Feet.	*Sec.-feet.*		*Feet.*	*Sec.-feet.*
Oct. 8. 6.30 a. m	0.66	780	Oct. 10. 12 m	6.93	31,450
1 p. m	1.50	3,175	1 p. m	6.95	31,650
6.30 p. m	2.17	5,500	2 p. m	7.13	32,800
8 p. m	2.59	7,300	3 p. m	7.19	33,150
10 p. m	3.00	9,125	4 p. m	7.25	33,500
11 p. m	3.33	10,700	5 p. m	7.39	34,450
12 p. m	3.50	11,525	6 p. m	7.39	34,450
9. 1 a. m	3.50	11,550	7 p. m	7.40	34,500
2.30 a. m	3.59	11,950	8 p. m	7.54	35,350
4 a. m	3.50	11,525	9 p. m	7.62	35,800
6 a. m	3.66	12,300	10 p. m	7.60	35,700
8.30 a. m	3.75	12,775	11 p. m	7.57	35,500
9.40 a. m	4.00	14,075	12 p. m	7.43	34,650
10.55 a. m	4.66	17,650	11. 1 p. m	7.47	34,950
12 m	4.75	18,200	2 a. m	7.5	35,100
1 p. m	5.25	21,050	3 a. m	7.42	34,700
2 p. m	5.37	21,750	4 a. m	7.3	34,450
3 p. m	5.45	22,250	5 a. m	7.3	34,150
3.45 p. m	5.37	21,750	6 a. m	7.3	34,150
4.25 p. m	5.29	21,300	7 a. m	7.37	34,300
5 p. m	5.23	20,950	8 a. m	7.33	34,100
5.45 p. m	5.19	20,700	9 a. m	7.31	33,900
6.30 p. m	5.17	20,600	10 a. m	7.23	33,450
7 p. m	5.11	20,250	11 a. m	7.25	32,525
8 p. m	5.13	20,350	12 m	7.18	33,100
9 p. m	5.17	20,600	1 p. m	7.18	33,100
10 p. m	5.21	20,750	2 p. m	7.17	33,300
11 p. m	5.27	21,150	3 p. m	7.08	32,450
12 p. m	5.4	21,950	4 p. m	7.00	31,950
10. 1 a. m	5.5	22,500	5 p. m	6.96	31,700
2 a. m	5.66	23,500	6 p. m	6.89	31,250
3 a. m	5.73	23,900	7 p. m	6.86	31,050
4 a. m	5.91	25,050	8 p. m	6.83	30,850
5 a. m	6.00	25,650	9 p. m	6.79	30,600
6 a. m	6.2	26,900	10 p. m	6.81	30,700
7 a. m	6.33	27,700	11 p. m	6.73	30,200
8 a. m	6.4	28,150	12 p. m	6.71	30,100
9 a. m	6.6	29,400	12. 1 a. m	6.63	29,600
10 a. m	6.83	30,750	2 a. m	6.59	29,350
11 a. m	6.89	31,250	3 a. m	6.55	29,100
11.35 a. m	6.97	31,750	4 a. m	6.51	28,800

Flow of Passaic River at Dundee dam, 1903—Continued.

Date and hour.	Gage.	Flow.	Date and hour.	Gage.	Flow.
	Feet.	*Sec.-feet.*		*Feet.*	*Sec.-feet.*
Oct. 12. 5 a. m.	6.42	28,250	Oct. 13. 9 p. m.	4.54	17,000
6 a. m.	6.42	28,250	10 p. m.	4.51	16,750
7 a. m.	6.39	28,100	11 p. m.	4.49	16,700
8 a. m.	6.39	28,100	12 p. m.	4.37	16,000
9 a. m.	6.25	27,200	14. 1 a. m.	4.37	16,000
10 a. m.	6.21	26,950	2 a. m.	4.35	15,925
11 a. m.	6.17	26,700	3 a. m.	4.35	15,925
12 m	6.05	26,100	4 a. m.	4.33	15,800
1 p. m.	6.06	26,050	5 a. m.	4.34	15,850
2 p. m.	5.93	25,200	6 a. m.	4.31	15,700
3 p. m.	5.89	24,950	7 a. m.	4.27	15,500
4 p. m.	5.87	24,800	8 a. m.	4.25	15,300
5 p. m.	5.79	24,300	9 a. m.	4.17	14,900
6 p. m.	5.77	24,150	10 a. m.	4.08	14,500
7 p. m.	5.75	24,250	11 a. m.	4.05	14,325
8 p. m.	5.73	23,950	12 m	4.02	14,150
9 p. m.	5.63	23,300	1 p. m	4.02	14,150
10 p. m.	5.59	23,100	2 p. m	4.01	14,100
11 p. m.	5.54	22,750	3 p. m	3.97	13,900
12 p. m.	5.49	22,450	4 p. m	3.94	13,750
13. 1 a. m.	5.44	22,200	5 p. m	3.85	13,300
2 a. m.	5.39	21,000	6 p. m	3.75	12,775
3 a. m.	5.35	21,650	7 p. m	3.75	12,775
4 a. m.	5.30	21,350	9 p. m	3.71	12,550
5 a. m.	5.24	21,000	12 p. m	3.66	12,300
6 a. m.	5.21	20,850	15. 6.30 a. m	3.50	11,525
7 a. m.	5.16	20,525	1 p. m	3.41	11,050
8 a. m.	5.13	20,350	6.30 p. m	3.41	11,050
9 a. m.	5.08	20,100	16. 6.30 a. m	3.00	9,125
10 a. m.	5.04	19,800	1 p. m	3.00	9,125
11 a. m.	5.00	19,560	6.30 p. m	2.91	8,700
12 m	4.94	19,200	17. 6.30 a. m	2.5	6,900
1 p. m.	4.89	18,900	1 p. m	2.5	6,900
2 p. m.	4.85	18,700	6.30 p. m	2.5	6,900
3 p. m.	4.84	18,650	18. 6.30 a. m	2.5	6,900
4 p. m.	4.75	18,200	1 p. m	2.41	6,500
5 p. m.	4.71	17,900	6.30 p. m	2.33	6,200
6 p. m.	4.66	17,650	19. 6.30 a. m	2	4,900
7 p. m.	4.64	17,550	1 p. m	2	4,900
8 p. m.	4.59	17,250	6.30 p. m	2	4,900

DAMAGES.

GENERAL STATEMENTS.

Estimates of flood damages are always approximations only. It is possible to determine with a fair degree of assurance the cost of replacing structures which have been carried away, to estimate the value of goods destroyed—especially if they be commodities stored in shops or warehouses—to calculate the amount of operatives' wages lost, and in the case of general mercantile business to estimate the damages incurred through consequent reduction of trade. Destruction by flood, however vast, is incomplete. It differs materially from destruction by fire, for often destructible property is of value after floods have passed. Buildings which are inundated still retain value, and many kinds of merchandise are not totally destroyed. Therefore when the amount of damages is calculated there is always to be taken into consideration the fact that a part of the material which has been flooded can be reclaimed, and retains some proportion, at least, of the value which it had previously possessed. Furthermore, damages by flood enter into practically every detail of social and business affairs. There are losses which are severe to one or more persons, and which can not be appreciated except by those whom the floods have actually overtaken. Therefore estimations of flood damages can be only approximate, and while a measure of accuracy may be reached with respect to a part of the losses, there remains a necessity for approximation which can not be classed with carefully computed damages along other lines.

HIGHLAND TRIBUTARIES.

Along the three northern tributaries, the Ramapo, Wanaque, and Pequanac, and at their confluence with the Pompton, the destruction by flood waters was far greater than along the Rockaway, Whippany, and upper Passaic, or in that area described as the Central Basin. In the drainage areas of the three tributaries last mentioned the waters were higher than in the flood of 1902, but the general effects were of the same nature, and consisted principally of flooded lands, houses, and washouts. There were few radical cases of complete destruction like those which marked the course of the flood in the northern tributaries. The principal interest is therefore confined to the Pompton and the three highland tributaries which discharge into it.

Ramapo River.—The greatest destruction was along the Ramapo. It is the largest of the upland branches, and was therefore the heaviest contributor to the main stream. Throughout the flood period the stream was especially violent, causing great apprehension in the lower valley.

The destruction along several stretches of the valley was almost complete. Nearly all the dams failed, and every bridge across the river, with one exception, was carried away. Some small villages were swept bare, and the damages to realty value and personal property were excessive.

It was only by strenuous measures that the dam impounding the waters of Tuxedo Lake was saved. If this had failed the destruction along the entire course of the river, even to the cities in the lower valley, would have been enormously increased.

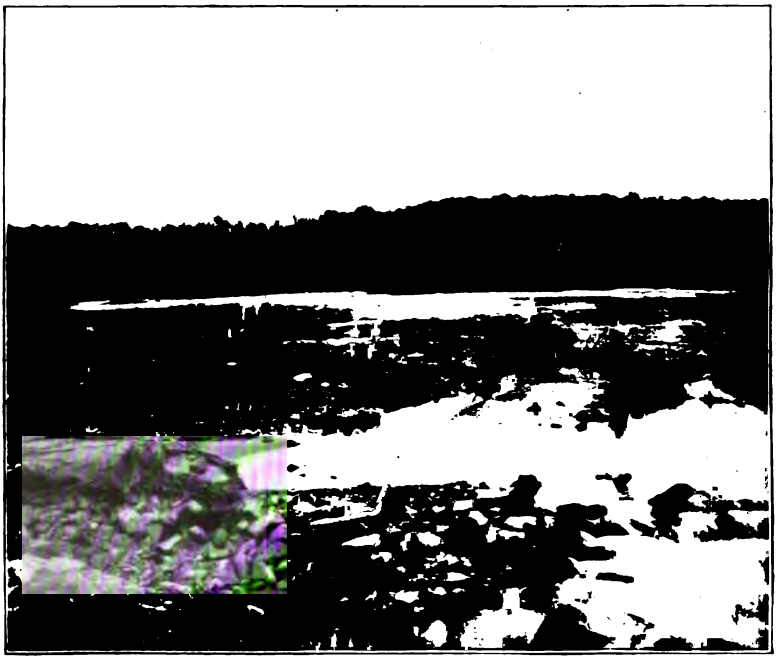
The dam at Cranberry Pond, in Arden, failed in the early part of the storm, the flood waters disabling the Tuxedo electric-light plant and inundating the Italian settlements along the river below. The failure of the dam conserving the waters of Nigger Pond, which lies at the head of a small tributary emptying into the Ramapo below Tuxedo, resulted in the inundation of Ramapo village. The village of Sloatsburg was practically obliterated.

The damage at Pompton Lakes was especially severe. During the early part of the flood the timber dam of the Ludlum Steel and Iron Company, which raised the water to a height of 27 feet, and afforded 7.04 horsepower per foot fall, was carried away with a part of the headrace. (See Pl. II, *A*.) This sudden emptying of Pompton Lake, an expanse of 196 acres (see Pl. II, *B*), was extremely destructive to Pompton Plains, and the destruction of the dams above on Ramapo River, which followed some time after the bursting of the lower dam, refilled Pompton Lake above its former level, and caused greater damage than that which resulted from the failure of Pompton dam itself. The large iron bridge just below the dam was carried away, with the stores of the Ludlum Steel and Iron Company. The river front along this company's property was destroyed, along with coal docks at the head of Morris Canal feeder. The channel of the river below the dam is filled with débris, which will raise the height of the water in the tailrace, and unless it is cleared will diminish the available power at the iron works. It has been authoritatively announced, however, that the power facilities will not be restored, as the Ludlum Steel and Iron Company is preparing to use steam power exclusively.

Pequanac and Wanaque rivers.—Along Pequanac River the principal damage consisted of washed-out roads and destroyed bridges. The large ponded area in this basin was practically full at the time of the flood, and, as measurements at Macopin dam show, the run-off per square mile was extremely large. In the Wanaque drainage area the storage facilities afforded at Greenwood Lake were probably useful in holding back a part of the water for a brief period, but the damages along the stream are comparable to those of the Pequanac.

The effect of the flow from these two streams, added to that of the Ramapo, was particularly disastrous over the Pompton Plains. Three

A. POMPTON LAKES DAM AND WATER FRONT OF LUDLUM STEEL AND IRON COMPANY.



B. DRY BED OF POMPTON LAKE.

FLOOD DISTRICT OF PATERSON, N. J.

bridges at Pompton station, over Wanaque and Pequanac rivers, were carried away, and in the end one bridge only remained over Pompton River, that at Pequanac station. In all about 100 houses were inundated on Pompton Plains, and the damage to roads and culverts was particularly severe.

The total loss in the drainage area of Pompton River was $350,000.

CENTRAL BASIN.

Over the Central Basin there was the usual impounding of flood waters, but the effects were not materially different from those described in the report on the flood of 1902. The damage along this basin from floods of this character is accumulative by reason of the fact that the presence of water over the land for so long a period kills the desirable feed grasses and fosters in their place the coarse meadow grass. This effect has been observed for some years, particularly since the flood of 1896. It is estimated that over the Central Basin the damage to crops and arable land alone arising from the floods of 1902 and 1903 amounts to $300,000. A statement of the damage arising from the later flood can not separately be made, as its effect upon the fertility of the meadow lands can not be determined without the experience of a planting season.

LOWER VALLEY.

The flow of the stream through the constricted channel at Little Falls and on to Great Falls at Paterson is given in the weir measurements on page 17. It was attended by comparatively large damages, the features of which were not materially different from those described in the previous report. The pumping station of the East Jersey Water Company, situated just below Little Falls dam, did not suffer as severely as during the previous flood, by reason of the fact that extensive and effective barricades were placed so as to keep a large part of the water away from the pumps. This was not accomplished in the flood of 1902. The total damage in this district amounted to nearly $200,000.

The channel contours were changed somewhat in this portion of the stream. In the river at the pumping station of the East Jersey Water Company there was completed a somewhat interesting cycle of changes, described in the following extract of a letter from Mr. G. Waldo Smith, chief engineer for the New York aqueduct commissioners, and formerly engineer and superintendent of the East Jersey Water Company:

"No better illustration of the old adage, 'The river claims its own,' could be given than that offered by the action of Passaic River at Little Falls, New Jersey, at the point where the works of the East Jersey Water Company have been constructed. These works

were built between 1897 and 1900. In the course of the work the river channel for a distance of several thousand feet down stream from the power house was drained and improved, so that the head on the wheels at the ordinary stage of the river was increased about 6 feet. From the time this improvement was completed to March, 1902, through the action of the ordinary flow of water and moderate floods, this head had been reduced about one-third. The great freshet of March, 1902, cut off about another third, and the recent flood has completed the cycle and entirely wiped out the benefit due to the river improvement, and the water at the pumping station stands now at almost precisely the same level that it stood before any improvements were undertaken. New bars were formed in approximately the same location as they existed before, and, so far as possible, except for the changed conditions brought about by the building of the power station, the condition of the river is not dissimilar to that existing when the work was commenced.

"In this connection it might be well to state that a New Jersey drainage commission, in blasting out a channel below the Little Falls dam some years ago, dumped a considerable portion of the excavation in the deep water under the Morris Canal viaduct.

"The action of the two great floods, March, 1902, and October, 1903, has washed a large part of this material out of this deep hole and piled it up in the river about 300 feet below where the river widens, and reduces the force of the current.

"I have made no estimate of the amount of material deposited in the river, but offhand should say that it would be at least 100,000 yards."

Paterson.—The flood district in the city of Paterson (see Pl. III) comprised 196 acres and involved the temporary obstruction of 10.3 miles of streets. Along the streets close to the river banks the height of water was 12 feet, sufficient to inundate the first floors of all the buildings (see Pl. I, *B*), and in some cases to reach to the second floor. During this flood period householders who remained at their homes were compelled to use boats, while in the more exposed places the danger was too great to admit of remaining, and at one time 1,200 persons were housed and fed in the National Guard armory at Paterson.

The bridges crossing Passaic River in Passaic, Essex, and Bergen counties were almost completely destroyed, and the damage amounted to $654,811. Within the limits of Paterson, below Great Falls, all of the highway bridges except two were either severely damaged or completely carried away. West street bridge, the first below the falls, was a Melan concrete, steel-arch structure, built in 1897, and costing $65,000. It was composed of three spans, each about 90 feet long. The flood practically split two spans longitudinally, the upstream side of each, equal to about one-third of the width of the bridge, being carried

A WASHOUT AT SPRUCE STREET, PATERSON, N. J.

B. RIVER STREET, PATERSON, N. J., AFTER FLOOD.

A. EFFECTS OF FLOOD IN MILL DISTRICT, PATERSON, N. J.

B. THE WRECK OF A HOTEL IN PATERSON, N. J.

away. This structure was built to conform to the established grades of streets on both sides of the river and was completely inundated, forming a barrier for floating débris and practically making a dam in the river. Main street bridge is a 3-span, steel-arch structure, which was completely covered during the flood, but was only slightly injured. Arch street bridge, built in 1902 to take the place of a structure carried away by the March flood, was a concrete-arch bridge of three spans. It was undermined at the north pier and collapsed, being practically destroyed. The original cost of this bridge was $34,000. Its piers presented a serious obstruction to the flow of the stream, especially as the channel is very narrow at this point. In addition to this, the bridge was of low grade and admirably adapted for deterring flood flow. Below Arch street bridge all the other structures crossing the Passaic were of iron and were carried away, with the exception of Sixth avenue and Wesel bridges. Those destroyed were designated as follows: Straight street, Hillman street, Moffat, Wagaraw, Fifth avenue, East Thirty-third street, and Broadway bridges. All these structures were built too low, and were inundated during the early stages of the flood.

The damage to real property, stock, and household goods in the city of Paterson amounted, according to certified returns, to about $2,700,000. It is impossible to secure correct figures, because merchants and manufacturers refuse to give details of losses, fearing that the publication thereof would affect their credit. General ideas concerning the destruction by the flood can be gathered from Pls. I, *B*, III, IV, V, and VI.

Passaic and vicinity.—Below the city of Paterson destruction was as complete as in Paterson, although the damage was not as great because the improvements were not as valuable. Damage to property, exclusive of public works, in this region, amounted to about $1,250,000. This estimate does not take into consideration losses by manufacturers arising from destruction of raw materials or finished products. The flood was about 4½ feet higher than that of 1902. (See Pl. VII, *A*.)

On the right bank of Passaic River, in the city of Passaic, the damage was severe, especially to manufacturing plants. In addition to the flood in the Passaic itself, the bursting of Morris Canal, a few miles east of Passaic, flooded Wesel Brook, which in Passaic is used as the tail-race of the Dundee Power Company. The capacity of Wesel Brook channel is limited, and the extraordinary amount of water which was turned into it carried away all culverts and bridges from Richfield to Passaic.

Below Passaic, along the river front of Essex County, the damages to bridges amounted to $50,000. (See Pl. VII, *B*.) The loss due to washouts in roads throughout the county amounted to $15,000. The

effects of the flood were apparent along the entire length of the river and into Newark Bay. The damage from inundation in Newark and vicinity amounted to $753,199.

The figures above given with reference to damage along Passaic River are uncommonly accurate, being for the most part the result of a house-to-house canvass by the northern New Jersey flood commission. As has been stated above, tradesmen are reluctant to give full details with reference to their losses through fear of injured credit. Roughly estimating the damage as a whole, and taking into consideration factors which were given to the writer confidentially, the damage throughout the drainage area from this flood will amount to not less than $7,000,000.

PREVENTIVE MEASURES.

GENERAL DISCUSSION.

In the consideration of means of preventing damages by floods every plan proposed falls under one of two general heads—the storage of flood waters or an increase in the capacity of the streams.

The first plan involves the construction at selected localities of reservoirs of sufficient size to hold all or a greater part of the waters which run over the surface during and after storms. This plan is not practicable except where valleys or plains are inclosed by high ridges and these ridges approach sufficiently near each other to admit of the economical construction of a bank or dam across the gorge or bed of the stream which flows through, so that the inclosure will be complete and form a water-tight basin. Where such a reservoir exists the water can be held back and gradually let down through properly provided gates so that the channel will not be flooded.

For flood purposes alone it would be necessary to provide reservoirs of sufficient capacity to contain the run-off waters resulting from the largest storms. With such provisions it would be necessary to entirely empty the reservoir as soon as possible after a storm had passed and leave its full capacity available for the next storm. It is therefore better, wherever possible, to provide a reservoir capacity considerably larger than that represented by the run-off from the heaviest storms, so that water may be stored for use as power or domestic supply. With such provision it is necessary merely to draw from the reservoir water to a depth equivalent to the stream run-off in the drainage area above.

The second plan for prevention of flood damages involves provisions for letting the flood water out rapidly by removing obstructions to its flow by straightening and deepening the channels and providing long embankments, dikes, or levees which rise above the ordinary river level to a height exceeding that of the stream during its highest floods. This plan is most generally followed in the case of large rivers like

A. DEVASTATION IN HEBREW QUARTER, PATERSON, N. J.

B. A COMMON EXAMPLE OF FLOOD DAMAGE.

A. INUNDATED LANDS AT PASSAIC, N. J.

B. UNDAMAGED BRIDGE ACROSS PASSAIC RIVER AFTER PARTIAL SUBSIDENCE OF FLOOD.

the Mississippi, where the contributing area is enormous and the conservation of the waters would be impracticable even if the nature of the country would admit of the construction of reservoirs. In Switzerland, where the torrents occasioned by the rapidly melting snows are especially destructive, the flood waters are confined by a series of parallel dikes on each side of the river, which have the effect of dividing the flow into several parallel streams. As the main river channel fills and overflows the inner dikes, the overflow water collects into the first series of parallel channels, and when a height is reached at which the second dikes are overflowed the water collects into the third, and so on. This gives an enormous carrying capacity, the limit of which is approached slowly, and therefore abundant opportunity is afforded for preparation upon the part of the riparian owner.

The drainage basin of Passaic River is admirably adapted to the development of the conservation system. At its headwaters in the mountains of northern New Jersey are numerous sites for reservoirs. The comparatively limited area draining into Passaic River makes such a scheme relatively inexpensive. On the other hand there is abundant opportunity for effective work in removing obstructions and straightening and deepening the channel of the lower river. So that, all things considered, the prevention of flood damages in the Passaic Basin can be best accomplished by a combination of the two general methods above outlined.

LOWER VALLEY IMPROVEMENTS.

The channel of Passaic River below Great Falls, at Paterson, is of limited capacity. To anyone making an inspection, especially within the city of Paterson, it is readily apparent that the river bed has for years been considered a legitimate field for encroachment. Owners of lands fronting on the river have increased their holdings by filling in beyond the channel line. Buildings have been erected upon these tracts and the builders have not hesitated to extend retaining walls still farther into the river bed. Refuse from the city's streets, light and unstable in character, has been freely deposited upon the bank to be carried out into the river. Thus the channel has been constricted laterally, the bottom raised, and there is left for the flood waters no alternative than that of extending themselves in the upward direction. It would seem that this, at least, should have been unobstructed. Such, however, is not the case.

The bridges across the Passaic have apparently been erected without reference to channel capacity. The authorities have evidently considered it more important to retain established approach levels than to provide proper capacity for river water. As an example the following instance may be cited: During the flood of 1902 a steel truss bridge across the river in Paterson was carried away. The point of

A. EFFECTS OF FLOOD IN MILL DISTRICT, PATERSON, N. J.

B. THE WRECK OF A HOTEL IN PATERSON, N. J.

The Pompton River has a drainage area above Mountain View of 420 square miles. It is formed near the head of the basin by the confluence of the Pequanac from the northwest, the Wanaque from the north, and the Ramapo from the northeast. * * *

The entire flow from this watershed may be stored by building a dam across the gap at Mountain View and converting Pompton Plains into a great lake covering an area of 21 square miles. The elevation of the river at the gap is 168 feet. The slopes in the basin being gentle up to an elevation of 220 feet and abrupt beyond it, it will be advisable to take this as the minimum or low-water level of our reservoir. It is generally estimated that 25 per cent of the volume of the mean annual rain on a given catchment is sufficient reservoir capacity to fully utilize the flood flows. We have long series of observations of rainfall at three points, which may be taken to fairly represent the Passaic catchment. At Newark the mean annual rainfall is 46.2 inches, at Paterson, 50 inches, and at Lake Hopatcong, 42. The last being on the Highlands, like most of our watersheds, is perhaps the safest to use. Now, 25 per cent of 42.5 inches, 10.62 inches, which, on 420 square miles, give a volume of 10,362,000,000 cubic feet, the necessary capacity of reservoir.

By raising our reservoir to 240 feet when full we secure a capacity of 10,493,000,000 cubic feet, or ample to utilize the heaviest floods of the watershed. This gives a beautiful sheet of water 21.1 square miles in area, with bold, rocky shores, and a depth at dam of 72 feet. We secure the above capacity by uncovering but 22 per cent of the reservoir bottom; and, as we shall presently see, we shall rarely need more than half this storage, and probably not oftener than once in ten years will we expose over 10 per cent of the area. By building side dams to keep certain flats always flowed this may be reduced to 5 per cent; and this area will be pretty evenly distributed around 36 miles of uninhabited shore line, leaving the reservoir open to no valid sanitary objections. On the contrary, by relieving the remainder of the Passaic Basin of the flood waters of the Pompton, which now flow large areas of flat land during wet seasons, the sanitary condition of the valley would be much improved.

In constructing this reservoir Mr. Vermeule stated that the following work would be necessary:

The removal of the Delaware, Lackawanna and Western Railroad from the basin by changing the alignment for 6 miles. It may be done without increase of length or detriment to the alignment.

Three and one-fourth miles of the Morris Canal must be rebuilt. No engineering difficulties are involved.

Of the New York and Greenwood Lake Railway, 9 miles would have to be rebuilt.

The New York, Susquehanna and Western Railroad would be slightly shifted or raised for 3¾ miles.

A dam 2,400 feet long and 80 feet in height, with tunnels, wasteweir, and accessory works would be required at Mountain View. The situation is such that an ample wasteweir may be built at a low-side dam on the solid rock of Hook Mountain remote from the dam, and outlets may be had by tunneling the same ridge. Hence the dam may be a plain, heavy earthen embankment; built, of course, with every precaution but subject to less than the usual dangers of such works. However, a masonry dam might readily be substituted.

There would be 14,000 acres of arable land, swamps, and rough mountain land flowed.

The works are estimated to cost as follows:

Railroad and canal diversions	$505,000
Dam and accessory works	1,162,000
Land damages	1,400,000
Total	3,067,000

A recomputation of the drainage area above Mountain View, made by the northern New Jersey flood commission, shows that it is 380 square miles in extent. It was decided by this commission that the construction of this reservoir would be the most approved method of preventing disastrous floods in the lower valley of the Passaic. By raising a dam to a height of 202 feet above tide, 8 inches of water on the drainage area above might be held back, which, it was believed, would be a sufficient maximum for flood catchment. With this amount of storage the estimates of the flood commission showed that the remainder of the drainage area would not turn a sufficient amount of water into the lower valley channel to cause flood damages.

It was also demonstrated by the flood commission that by increasing the height of the dam an opportunity would be afforded for conserving water, and at the maximum height of 220 feet above tide sufficient storage capacity would be available to provide 5,000 horsepower at Little Falls, Great Falls, and Dundee dam throughout all dry seasons. The value of such a storage reservoir for municipal water-supply purposes is self-evident.

The cost of Mountain View reservoir would be about $3,340,000. Developed for flood catchment with the spillway of the dam at 202 feet above sea level the area of the reservoir would be 13.4 square miles, and the storage capacity 7,200,000,000 cubic feet.

RAMAPO SYSTEM.

Along the Ramapo Valley there are alternative propositions, one of which involves the construction of a dam below Darlington and another across the head of Pompton Lake.

In either case the water might be raised to the 300-foot contour, and if the dam across Pompton Lake were constructed a continuous lake would be formed extending 10½ miles to Hillburn, N. Y. The improvement in either case would be positive, for as the country surrounding is hilly or mountainous it affords excellent opportunity for the location of summer homes and parks, the lake being a potent factor in beautifying the situation and increasing the value of the surrounding region. There are, nevertheless, several things to be taken into consideration, the most important of which are the improvements which have been made by wealthy residents along the valley where it has already been developed as a summer resort.

By the construction of a dam at Darlington 1,100 feet long and 70 feet high, the water would be raised to the 300-foot contour. The reservoir would have a water area of 2,064 acres, and the approximate storage capacity of 2,325,000,000 cubic feet.

A dam across the head of Pompton Lake 2,850 feet long and 100 feet high would raise the surface of the proposed lake to the 300-foot contour. This reservoir would have an area of 6.19 square miles and

a capacity of 6,300,000,000 cubic feet, equal to 17.5 inches run-off from the drainage area. Here the measure of safety is wide, and if there were drawn from the lake an amount of water equal to 12 inches on the drainage area there would still be 5.5 inches which could be used for compensating purposes.

The construction of either one of the above-described reservoirs would involve interstate complications, as the 300-foot contour in Ramapo Valley includes a considerable part of the State of New York. This obstacle was deemed insurmountable by the northern New Jersey flood commission, and that commission directed studies to a reservoir which at the time of maximum flood would not back water into New York State to a greater height than it already rises during such floods. The following description is taken from the report of the engineering committee of the flood commission:

An admirable dam site is offered on Ramapo River about 2 miles above Oakland village. The drainage area tributary to this point is about 140 square miles in extent, the country for the most part being quick-spilling and upland. By constructing there a dam 700 feet long and 65 feet high a reservoir with a water surface of 2.8 square miles would be afforded, the flow-line elevation being 280 feet above tide. The capacity of such a reservoir would be 1,768,000,000 cubic feet, equal to about 5.5 inches on the drainage area.

WANAQUE SYSTEM.

Near the headwaters of Wanaque River is Greenwood Lake, a large body of water described in Water-Supply Paper No. 88. Its value as a flood catchment basin is somewhat uncertain, as it is used as a storage feeder for Morris Canal. The surface level of this lake is controlled by gates, which naturally are operated by the canal authorities for the benefit of the canal. Therefore it is the object to store as great a volume of water as possible, and the water falls below the dam crest at the outlet of the lake only when the dam opens in dry seasons and makes it necessary Under such conditions there is no certainty that storage capacity will be available during the time of a great storm, and in fact Greenwood Lake has been overflowing at the commencement of the storms which caused both of the recent floods.

In view of the condition expressed above it will be necessary in providing for flood catchment in the Wanaque drainage area to omit entirely from consideration the possibility of assistance from Greenwood Lake. Below this point in the basin are several sites at which could be raised dams, which would effectually retain a large proportion at least of storm run-off. They may be described as follows:

Midvale reservoir.—By building a dam 60 feet high and 1,200 feet long across Wanaque River near Midvale, a reservoir would be formed which would have a water surface of 2.1 square miles and a capacity of 1,491,000,000 cubic feet. The drainage area above this site is 83 square miles, and the storage capacity would therefore be equal to about 7.7

inches on the drainage area. The construction of this project would involve the relocation of about 4½ miles of the New York and Greenwood Lake Railroad; the damages apart from this would be nominal, the cost of the entire reservoir construction being about $1,000,000.

Ringwood reservoir.—Ringwood Creek runs through a gorge about 1 mile above its confluence with the Wanaque. Above this is a well-defined basin. A dam about 70 feet high and 585 feet long would create a lake having an area of 520 acres, the surface of which would be 380 feet above sea level. The drainage area tributary to this point has an area of about 20 square miles, and as the proposed reservoir would have a capacity of 915,800,000 cubic feet, there could be conserved a run-off of 20 inches. Allowing for a flood run-off of 12 inches there would still be available for compensating purposes 8 inches on the basin, equal to 373,550,000 cubic feet. The construction of this reservoir would involve the relocation of about 2 miles of the Ringwood branch of the New York and Greenwood Lake Railroad, and the condemnation of comparatively valuable improvements in the proposed basin.

West Brook reservoir.—The drainage from 5.7 square miles might be conserved by the erection of a dam on West Brook, a tributary of Wanaque River, which enters it from the west. There is an available site at which a dam 280 feet high might be erected. At this elevation the length along the top would be about 1,150 feet and about 2,330,000,000 cubic feet of water would be impounded. Little benefit would be derived from such a reservoir, as the limited drainage area affords a comparatively small proportion of flood run-off that might be well cared for at a lower point. For compensating purposes, however, a reservoir might be constructed here, the capacity of which could be adjusted to the actual demands. If the dam were raised to a height of about 280 feet from the base the storage afforded would be equal to 176 inches on the watershed, or about four average years of precipitation, which is far beyond all probable storage necessities. The maximum available storage capacity is given in this case merely to show possibilities.

PEQUANAC SYSTEM.

There are few available reservoir sites of large size along the lower reaches of Pequanac River. In the upper basin, however, there is a sufficient available storage capacity to afford almost complete control of destructive floods from that part of the drainage area. Large tracts are already reserved by the city of Newark for collection of municipal supply, and the storage capacity developed is sufficient to serve the city throughout the driest seasons. The total capacity of Clinton, Oakridge, and Canistear reservoirs is about 1,155,000,000 cubic feet. These basins are not available for flood catchment, as the water is used for city purposes and an endeavor is made to have in storage at all

times the largest possible amount. The condition is exactly similar to that described in the case of Greenwood Lake. In considering the means for the construction of flood-catchment reservoirs in Pequanac Basin there must be taken into account the conservation and delivery of the Newark supply. The adjustments with reference to the amount of water available at Macopin intake would have to be met, and if the system were interfered with compensation therefor would be taken into consideration.

Newfoundland reservoir.—Pequanac River passes through a deep gorge between Copperas and Kanouse mountains, just below the village of Newfoundland. This point has been considered an excellent site for the construction of a dam, and in the installation of the present water-supply system of Newark it is proposed that the entire valley in which Newfoundland is situated be overflowed. The site is one of the most advantageous known for the creation of a flood-catchment basin. If a dam 50 feet high were erected across this gorge, a lake would be formed which would have a surface area of 3.15 square miles and a capacity of 3,267,200,000 cubic feet, equal to a storage of about 30.5 inches on the 46.12 square miles of contributing drainage area. This would afford complete protection in case of a sudden run-off of 12 inches, would provide for the supply of the city of Newark without greatly disturbing the present storage system of that city, and would still yield a large amount of water for compensating purposes in dry seasons.

The construction of Newfoundland reservoir would be very expensive, as it would involve the flooding of Newfoundland Village, in which there is considerable improved property. About 3 miles of the track of the New York, Susquehanna and Western Railroad would be submerged, as well as a considerable mileage of macadamized highways. On the whole, however, the Newfoundland reservoir project is the most favorable which can be found on the Pequanac Basin. There are above this point numerous reservoir sites, but their combined capacity would not be equal to that of the proposed Newfoundland reservoir, and the construction would be probably quite as expensive.

Stickle Pond reservoir.—Below Newfoundland there are few available places at which water could be stored. Stickle Pond is probably the best adapted of any of those available. If a dam 1,050 feet long and 80 feet high were erected across the river about 1 mile below the present outlet of Stickle Pond, a lake would be formed having a surface area of 422 acres and a storage capacity of about 800,000,000 cubic feet. The drainage area above this dam would be approximately 4 square miles. This is a comparatively small amount of storage, yet it would provide for all flood catchment in that comparatively limited area and would be of assistance at times in compensating the dry flow of the Pequanac.

ROCKAWAY SYSTEM.

Rockaway River offers a greater number of available reservoir sites than either of the other highland tributaries of the Passaic. Some of the reservoirs which could be constructed could be used solely as catchment areas to hold back flood waters, while the capacity of others would be so much greater than any single flood run-off that they might serve also as compensating reservoirs. A large dam is now in process of erection at Old Boonton, conserving a considerable amount for the water for the municipal supply of Jersey City. This reservoir can not be depended upon as a flood-catchment area, as it will be the aim of those in authority to maintain the water in it as high as possible.

Powerville reservoir.—A short distance above Boonton the erection of a comparatively small dam would flood a large, irregular, flat basin having an area of a little more than 4½ square miles and extending up the Rockaway Valley to Rockaway Village, up Beaver Brook to Beech Glen, and north and south for considerable distances. The probable capacity of this reservoir has been estimated, and it is fairly certain that it is considerably more than would be sufficient for flood catchment. Its construction would, moreover, improve the entire valley and be of advantage to many interests.

The northern New Jersey flood commission has selected for investigation a reservoir site on Rockaway River at Powerville. By the erection of a dam across the stream at this point, 28 feet in height and 470 feet long, a reservoir 4.6 square miles in area, with a capacity of 1,565,000,000 cubic feet, would be afforded. The drainage area above this point is 114 square miles. The cost of such a reservoir is estimated at $600,000.

North from Powerville, near the confines of the proposed Powerville reservoir, there is an available reservoir site along Stony Brook. By the erection of a dam 1,100 feet long and 120 feet high a lake would be formed 645 acres in extent, which would serve as a flood-catchment basin and a compensating reservoir. This reservoir would hold approximately 850,000,000 cubic feet. The construction of a reservoir at this place offers no engineering difficulties, and the project may be regarded as extremely favorable.

Dixons Pond, west of Rockaway Valley and northwest of Powerville, is a small sheet of water which lies in a valley which might be flooded to a greater height. By the erection of a dam 450 feet long and 30 feet high a lake of 136 acres would be created, which would form a part of the flood catchment and compensating service.

Longwood Valley reservoir.—A large storage basin is afforded in Longwood Valley which, if developed to its full extent, would extend from a point about a mile below Lower Longwood 7 miles up the headwaters and reach to about 1½ miles above Petersburg. An alternative proposition is afforded which involves the submerging of less than half this area.

A dam 800 feet long and 55 feet high might be erected across a gorge about 1 mile south of Petersburg. There would be formed a lake of about 1.247 square miles, or 800 acres in extent. The hamlet of Petersburg would be submerged, but the damages from the destruction of improved property would not be very great, as the improvements and the land are not especially valuable. This reservoir would have a capacity of about 964,000,000 cubic feet and the surface would be at a height of 800 feet above sea level.

The alternative plan, that of using a longer stretch of the valley for reservoir purposes, would involve the construction about 1 mile below Lower Longwood of a dam 1,300 feet long and 110 feet high. The reservoir thus formed would be 1,900 acres in extent and contain approximately 3,447,000,000 cubic feet. The drainage area above this dam is limited, and if the reservoir were drawn down to an amount equivalent to 15 inches upon the drainage area there would still remain an enormous amount of water which could be used in a compensatory way to tide over dry seasons.

Splitrock Pond.—By erecting a dam 550 feet long and 30 feet high across a gorge at the outlet of Splitrock Pond, a lake could be formed having an area of 625 acres and adding to the present storage capacity of the lake an amount approximately equal to 475,000,000 cubic feet, equivalent to 38.75 inches on the drainage area.

Thus it is seen that if this reservoir were drawn down an amount equivalent to 15 inches on the drainage area, which would without doubt give sufficient protection from all floods, there would still remain a storage capacity of 23.75 inches for compensating purposes in addition to the amount now available in Splitrock Pond. This project is one of the most attractive in the Rockaway Basin, as the damages which would be caused by flooding would be, comparatively speaking, nil. The property is, however, now owned by the East Jersey Water Company, and is prized highly as a reservoir site by that corporation.

UPPER PASSAIC BASIN.

Millington reservoir.—There is an area of swamp land, comprising a part of the drainage area of upper Passaic River above Millington, which is known as Great Passaic Swamp. It is bounded on the south by a long, narrow trap ridge known as Long Hill, the summit of which ranges from 400 to 500 feet in elevation, or roughly 200 feet above the border of this swamp. To the northwest the land rises gradually toward Trowbridge Mountains, while to the northeast is the terminal moraine. The outlet of Passaic River at Millington is by a narrow gorge, which offers natural facilities for the erection of a dam.

The whole situation is exceptionally good, and the surface of a reservoir might be fixed at any elevation between 240 and 300 feet above sea level. With the surface of the reservoir at 300 feet a dam 1,600 feet long and 90 feet high would be required. This lake would

have an area of 28.46 square miles. The drainage area above Millington has, however, an area of only 53.6 square miles, and the proposed reservoir would therefore cover more than half of this. Therefore the conservation of so large a quantity of water would not be necessary nor advisable, unless the beautifying of the surrounding country were an object to be taken into consideration, which might be profitable.

A better project, however, would be to construct a dam at Millington 900 feet long and 50 feet high, the crest being about 260 feet above sea level. There would be formed a lake with an area of 19.41 square miles, and a capacity of 1,477,600,000 cubic feet, equal to 9.864 feet on the drainage area. This project is too great for the necessities here presented, and would not be wisely considered unless it were found advantageous to improve the country generally as a place of suburban residence. The land which would be flooded with the reservoir crest at 260 feet is of a wet, swampy character, and its value for agricultural purposes is somewhat doubtful. Such construction would involve the flooding of 13 miles of road, which, however, would not involve a great loss of invested capital, as the roads generally are of a poor character.

A second alternative would involve the construction of a dam across the Millington gorge, 550 feet long and 30 feet high, raising the water to 240 feet above sea level and creating a lake of 14.40 square miles. This would conserve 4,026,000,000 cubic feet, equal to 2.69 feet on the drainage area. This would be ample for flood purposes and would still afford a large impounded area, as the drawing off of an amount equal to 10 or even 15 inches on the watershed would not reduce the size of the lake to any great extent.

The whole project here presented involves few difficulties, and as the drainage area above is of small extent, the mere question of conserving the flood waters could be met without great difficulty. The natural advantages, however, are so great and the land included within Great Passaic Swamp is of so little value that the surrounding country would be improved and beautified by the construction of such a reservoir. The opportunity for varying the character of the reservoir to meet the ideas of those interested seems unexampled, and as a whole it presents an extremely interesting field which may be profitably exploited.

SADDLE RIVER.

This stream has been described in the report on the flood of 1902, already referred to. It contributes a large amount of water to the main artery of the Passaic below Dundee dam, and as the river channel at that point is overburdened under the present conditions because of lack of slope and numerous catchments, together with what is known as the Wallington Bend, it increases very materially the damage caused by floods.

The most effectual remedy in the case of Saddle River floods is that of construction of flood catchments. No studies have been made of the situation in the Saddle River drainage area, but a superficial inspection of the basin shows that opportunities for the construction of flood-catchment reservoirs are not numerous.

SUMMARY OF FLOOD-CATCHMENT PROJECTS.

By following the plans described in the preceding pages absolute flood catchments may be provided above Little Falls on the Passaic Basin for 551.7 square miles, leaving only 221.2 square miles from which flood run-off would flow immediately. The accomplishment of this would involve the construction of Pompton reservoir, which would withhold all flood waters from the northern tributaries. It would leave unprovided for 20.2 square miles on the Rockaway, 71.7 square miles on the Whippany, 46.2 square miles on the upper Passaic, and 83.7 square miles tributary to the Central Basin and not included above.

Leaving Pompton reservoir out of consideration, and conserving flood run-off on the Ramapo, Wanaque, and Pequanac rivers, there would be absolute flood catchment up to a 12-inch run-off over 494.8 square miles above Little Falls. This would leave 278.1 square miles unprovided for, the run-off from which would not overburden the channel in the lower valley, provided, of course, that channel were improved to a maximum carrying capacity.

PREFERABLE RESERVOIR SITES.

The following table and discussion of preferable sites for flood prevention are taken from the report of the engineering committee of the northern New Jersey flood commission:

Table showing detailed facts regarding possible reservoir sites on Passaic drainage basin.

Reservoir.	Area of watershed.	Area of reservoir.	Height of dam.	Length of dam.	Elevation of flow line.	Storage, watershed.	Storage capacity.	Total cost.
	Sq. mi.	*Sq. mi.*	*Feet.*	*Feet.*	*Feet.*	*Inches.*	*Million c.f.*	
Ramapo	140	2.8	65	1,700	280	5.5	1,768	$900,000
Wanaque	83	2.1	60	1,200	275	7.7	1,491	1,000,000
Newfoundland	52	1.8	40	430	780	8	966	1,800,000
Rockaway	114	4.6	28	470	520	6	1,565	600,000
Millington	56	15.8	25	220	245	31	4,060	370,000
Great Piece	773	37	21	1,500	178.5	[a] 9	8,950	2,625,000
Mountain View	380	13.4	42	2,150	202	8	7,200	3,340,000
Do	380	13.9	44	2,380	204	9	7,900	3,460,000
Do	380	14.3	46	2,470	206	10	8,700	3,590,000
Do	380	17.4	60	3,000	220	17	15,000	5,260,000

[a] Including water discharged through fixed openings, in a flood similar to that of October, 1903. Maximum discharge, 12,000 cubic feet per second.

With the exception of the Millington reservoir site where the cost of the dam is a small factor, the elevation of flow line in the various reservoirs which determines the capacity was fixed so as to afford an approximate storage equal to a run-off of about 8 inches from the drainage area above each dam site. This amount is somewhat in excess of the run-off for the flood of October, 1903. It was found impracticable on the Rockaway reservoir site to provide for a storage greater than 6 inches. On the Wanaque the amount which can be stored falls slightly under 8 inches, while on the Ramapo it is possible to obtain only 5½ inches, by reason of the fact that with a greater storage capacity the slack water would reach into New York State. The economical height for a dam at the lower end of the Great Piece Meadow, if such dam is provided with fixed discharge openings which will carry a maximum outflow of 12,000 cubic feet per second, will provide a reservoir which will dispose of a run-off of 9 inches on the drainage area above.

The following combinations of reservoir sites, with their respective drainage areas, proportional storage, and estimated costs, give the facts necessary for final deductions:

Site.	Drainage area.	Water collected.	Equivalent area retarded.	Cost.
	Square miles.	*Inches.*	*Square miles.*	
Ramapo	140	5.5	96.25	$900,000
Wanaque	83	7.7	80	1,000,000
Pequanac	52	8	52	1,800,000
Rockaway	114	6	85.5	600,000
Total	389		313.75	4,300,000
Ramapo	140	5.5	96.25	900,000
Wanaque	83	7.7	80	1,000,000
Rockaway	114	6	85.5	600,000
Millington	56	31	56	370,000
Total	393		317.75	3,870,000
Great Piece	773	4.5	435	2,625,000
Mountain View	380	8	380	3,340,000

The necessity to retard the flow of or provide storage for approximately 380 square miles of highland drainage area has been determined after careful study, and there has been deduced an amount which may safely be expected to represent the maximum for the highest floods. When the highland tributaries are sufficiently checked the natural storage on Great Piece Meadow in its effect upon flood control becomes more apparent. Our investigations show that the holding back of the flood flow—that is, 8 inches run-off on approximately 380 square miles of flashy drainage area above Great Piece Meadow—is necessary to reduce the discharge in the river through the city of Paterson to 14,000 cubic feet per second for a flood similar to that of 1903.

From the foregoing table, in which different reservoir projects are compared, it is seen that only the reservoirs designated as Great Piece and Mountain View will fulfill the requirements within a reasonable limit of cost. It is also shown that a combination of any other available sites would involve the expenditure of more money for their construction and the control of less tributary drainage area than is fulfilled by the demands of the Passaic drainage basin. We are therefore brought to

the conclusion that only two of the projects above set forth will be effective. First, the construction of a regulating dam on the main stream above Little Falls, which we have called the "Great Piece" Meadow Reservoir, and second, the building of a dam at Mountain View across Pompton River. The relative cost of these reservoirs, constructed for flood control exclusively, is $2,625,000 for that on Great Piece Meadow and $3,340,000 for the Mountain View site. Details of these estimates are as follows:

Estimate of cost of Great Piece Reservoir, dam at Little Falls.

[Elevation of flow line, 178.5 feet. Storage and disposal of 9 inches collected.a]

Earth excavation, 17,600 cubic yards, at 35 cents	$6,160
Rock excavation, 8,800 cubic yards, at $2	17,600
Rubble masonry, 29,100 cubic yards, at $5	145,500
Ashlar masonry, 1,800 cubic yards, at $12	21,600
Facework of rubble masonry, 2,850 square yards, at $1.50	4,275
Concrete masonry, 250 cubic yards, at $6	1,500
Slope paving, 300 cubic yards, at $2	600
Crushed stone, 150 cubic yards, at $1.50	225
60-inch cast-iron pipe in place, 360 tons, at $35	12,600
Relocation of railroads, Erie, 5 miles, at $20,000; Delaware, Lackawanna and Western, 4.5 miles, at $40,000	280,000
Relocation of highways	170,000
Real estate:	
Above Mountain View	500,000
Additional for village of Singac	100,000
22,000 acres, at $50	1,100,000
	2,360,000
Add for engineering and contingencies	240,000
	2,600,000
Protection of pipe lines, Newark and Jersey City	25,000
	2,625,000

The effectiveness of a reservoir built upon the lines proposed in the case of Great Piece Meadow depends upon the adjustment of outflow so that the channel below will not be overborne, while at the same time sufficient storage capacity is afforded to hold temporarily the water which enters above the dam in amount greater than the carrying capacity of the outflow apertures. The dam across Passaic River above Little Falls would be provided with apertures which would discharge 12,000 cubic feet per second under the maximum head in the storage basin. As the flood rises these apertures would discharge a constantly increasing amount of water to the maximum, and for a considerable time thereafter the maximum would be maintained, the discharge decreasing after the flood according to the height of water remaining in the reservoir.

a Includes water discharged through fixed openings for a flood similar to that of October, 1903. Maximum flow, 12,000 cubic feet per second.

Estimated cost of Mountain View Reservoir.

[Elevation of flow line, 202 feet. Storage of 8 inches on watershed.]

Earth excavation:	
Stripping dam base, 83,500 cubic yards, at $0.30	$25,050
Core wall trench, 24,900 cubic yards, at $1	24,900
Rock excavation, 10,100 cubic yards, at $2	20,200
Rock fill in dam, 197,000 cubic yards, at $1.25	246,250
Rubble masonry, 23,200 cubic yards, at $5	116,000
Concrete, 30,000 cubic yards, at $6	180,000
Gate chambers and tunnels	65,000
Reconstruction of highways	142,400
Reconstruction of railroads	815,000
Real estate	1,360,000
	2,994,800
Engineering and contingencies	325,200
	3,320,000
Protection of Newark pipe line	20,000
Total cost	3,340,000

[Same for elevation of flow line, 204 feet. Storage of 9 inches on watershed.]

Earth excavation:	
Stripping dam base, 85,200 cubic yards, at $0.30	$25,560
Core wall trench, 26,000 cubic yards, at $1	26,000
Rock excavation, 10,600 cubic yards, at $2	21,200
Rock fill in dam, 214,000 cubic yards, at $1.25	267,500
Rubble masonry, 24,500 cubic yards, at $5	122,500
Concrete, 30,500 cubic yards, at $6	183,000
Gate chambers and tunnels	65,000
Reconstruction of highways	142,400
Reconstruction of railroads	815,000
Real estate	1,435,000
	3,103,160
Engineering and contingencies	336,840
	3,440,000
Protection of Newark pipe line	20,000
Total cost	3,460,000

The final recommendation of the committee involves the consideration of two projects for flood storage, one on Great Piece Meadow and the other above Mountain View on the Pompton. In making such recommendations the committee is of the opinion that it must take into account matters of engineering policy with regard to future needs and contingencies, as well as the bare necessities of the present.

If there were none other than the single problem of prevention the committee would advise the construction of the reservoir on Great Piece Meadow by reason of its smaller probable cost and its equal efficiency. It is plain, however, that there are many important features of public policy involved in the subject at hand. Population in the valley of the Passaic is developing so rapidly that in only a few years the present sources of water supply will be inadequate. The whole subject of water supply for northern New Jersey demands immediate consideration, and it would not be wise to take up the matter of prevention of flood damage in the Passaic without

basing the value of every project upon its adaptability for use in future water-supply needs.

By expending $2,600,000 a great reservoir could be constructed upon Great Piece Meadow which could not be adapted for any purposes except to regulate floods; it would stand in season and out of season a huge feature of the valley and entirely useless and inoperative save on the occasion of high water. However great might be the needs of the inhabitants of the Passaic Valley for a conserved water supply, the construction on the meadows, representing an enormous expenditure, would furnish no solution of the problem. It would admit of no enlargement for water-supply storage and would be available for no purpose except flood regulation.

When we consider the Mountain View project, however, we find that as a measure for the prevention of flood damages it fulfills all the requirements and provides in addition all the possibilities and advantages demanded inevitably in the near future. The Mountain View site is an ideal one for the reservoir, and its initial development for flood catchment does not involve the expenditure of a dollar that would be lost in the development of the basin to greater capacities for water supply. From its lowest level, at 202 feet above tide, to its maximum capacity, at a level of 220, there would be no depreciation. Every dollar spent in the initial construction would be effective in the maximum development.

The probable cost of Mountain View reservoir, estimated at $3,340,000, exceeds that of Great Piece by $700,000. It is realized that to many persons this margin may seem very wide. Let us consider briefly just what it really represents.

Suppose, for example, that the Great Piece project is constructed at a cost of $2,600,000. After the elapse of a few years it will be necessary to provide additional storage in the Passaic highlands for water supply or the maintenance of water power. The Mountain View reservoir, or its equivalent in capacity and cost, will then be necessary. The situation will then be as follows: By constructing the Great Piece reservoir in preference to the Mountain View for flood catchment, $700,000 would be saved. We can consider that this amount might be expended to pay a part of the cost of additional conservation above referred to. If, on the other hand, Mountain View had been constructed, there would have been paid on the final cost of conservance the sum of $3,340,000, which, as stated in previous pages, would also have effected flood relief. There would then be the difference between $2,600,000 and $700,000, or $1,900,000, which represents the actual loss which would accrue by reason of the construction of Great Piece reservoir.

The engineering committee, after presenting the merits of both Great Piece Meadow and Mountain View projects, therefore recommends the adoption of the latter in spite of its greater cost, because it is believed that in the end the construction of the Great Piece project would involve an expenditure not warranted by public economy or general expediency.

GENERAL CONCLUSIONS.

1. Great floods in the Passaic Basin arise only after a specially violent precipitation.

2. Under present conditions floods may be expected at frequent intervals.

3. A part of the damage along the lower valley is the result of encroachments on the part of individuals and public and private corporations.

4. The channel in the lower valley may be improved at certain points by straightening it and judiciously making cut-offs.

5. Without the construction of numerous levees the lower valley channel can not be made to carry great flood waters without damage.

6. Immunity from floods can be effected only by the construction of catchment reservoirs in the highlands or levees in the lowlands.

7. Levee construction would involve more damage than is now caused by floods, and the cost thereof would be prohibitive.

8. Flood catchment reservoirs may be constructed economically and provide storage to compensate for the dry-season flow, thereby maintaining water power at Paterson, Passaic, and other points, and providing for municipal water supply in the future.

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DEPARTMENT OF THE INTERIOR
UNITED STATES GEOLOGICAL SURVEY
CHARLES D. WALCOTT, DIRECTOR

PROCEEDINGS

OF

FIRST CONFERENCE OF ENGINEERS OF THE RECLAMATION SERVICE

WITH ACCOMPANYING PAPERS

COMPILED BY

F. H. NEWELL
CHIEF ENGINEER

WASHINGTON
GOVERNMENT PRINTING OFFICE
1904

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
Washington, D. C., September 28, 1903.

SIR: I have the honor to transmit herewith a manuscript relating to the conference of the engineers of the reclamation service held at Ogden, Utah, September 15 to 18, 1903.

The necessity for this meeting arose from the fact that, while the reclamation service had been in active operation for over a year, its personnel was largely made up of men who were new in Government service, and unacquainted with each other and with the conventional methods of transacting business. As projects in each State had reached a point where relative merits must be considered, it was deemed advisable to bring the principal engineers together, in order to discuss somewhat informally the methods and results of work.

The place and time of this meeting was determined by the facts that Ogden is a central point within the reclamation States, and that the Eleventh Irrigation Congress was then held there. At this congress delegates were present from the thirteen States and three Territories named in the reclamation law, as well as from Texas and the country farther east. This enabled the engineers of the reclamation service to meet public men and others who are interested in the work and to exchange views freely concerning it.

About twenty-five of the principal consulting, supervising, and district engineers of the reclamation service met at Ogden, this being about one-tenth of the number of engineers and assistants engaged in the field work of the reclamation service. The headquarters were conveniently located, and the engineers met in sessions beginning early each morning and continuing, with short interruptions, throughout the day and evening, discussing matters of organization, personnel, methods of work, cost of surveys and construction, details of plans, specifications, etc. From time to time prominent men or delegations from various States were received, and brief addresses were made to the engineers by governors, senators, and others. A partial stenographic report was made of the proceedings, and this has been

amplified in the accompanying manuscript. It is believed to be desirable to preserve this material and to send it out to the various engineers and assistants who were not present at the meeting, in order to give them information concerning some of the subjects discussed. The public also is interested to a certain extent in many of these details, and it is considered permissible to present, in this somewhat informal discussion, matters that might not be pertinent to the annual report.

I therefore respectfully recommend that this matter, together with such incidental information as may be convenient for reference in this connection, be printed as one of the water-supply and irrigation papers.

Very respectfully,

F. H. NEWELL,
Chief Engineer.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

PROCEEDINGS OF FIRST CONFERENCE OF ENGINEERS OF RECLAMATION SERVICE, AT OGDEN, UTAH, SEPTEMBER 15–18, 1903.

Compiled by F. H. NEWELL, *Chief Engineer.*

CALL FOR CONFERENCE.

The following notice was sent out from the office of the chief engineer of the reclamation service:

UNITED STATES GEOLOGICAL SURVEY,
RECLAMATION SERVICE,
Washington, D. C., July 15, 1903.

NOTICE.

The various engineers and assistants of the reclamation service will assemble at Ogden, Utah, on September 14, to hold a conference on important details during the days from September 15 to 18, inclusive. It is expected that, as far as possible, the time will be devoted to official business and the discussion of plans and results, and adjournment will be taken to attend the meetings of the National Irrigation Congress.

The following-named persons are expected to be present: Messrs. Ahern, Babb, Bien, Blanchard, Chandler, Darton, Davis, Fellows, Field, Fitch, Hinderlider, Lippincott, Matthes, McConnell, Newell, Noble, Peery, Quinton, Reed, Ross, Sanders, Savage, Storrs, Swendsen, Taylor, Walter, Whistler, and Wisner.

On Friday morning it is expected that the engineers will participate in the meeting of the National Irrigation Congress.

Very respectfully, F. H. NEWELL, *Chief Engineer.*

ATTENDANCE AT CONFERENCE.

In accordance with the above notice, a conference was held at Ogden, Utah, at the time of the meeting of the Eleventh Irrigation Congress, September 15 to 18, 1903. The following persons were present:

Ahern, Jeremiah, district engineer, northern Wyoming.
Babb, Cyrus C., district engineer, Montana.
Bien, Morris, engineer in charge of land and legal matters, Washington, D. C.
Chandler, A. E., hydrographer and State engineer of Nevada.
Darton, N. H., geologist in charge of western section of hydrology.
Davis, Arthur P., supervising engineer for Arizona, etc.
Fellows, A. L., district engineer for Colorado.
Field, John E., district engineer for North Platte, Wyoming and Nebraska.

Fitch, Chas. H., supervising engineer for South Dakota, etc.
Lippincott, J. B., supervising engineer for California.
McConnell, I. W., engineer on Gunnison Tunnel.
Newell, F. H., chief engineer, Washington, D. C.
Noble, T. A., district engineer for Washington.
Peery, E. H., law clerk.
Quinton, John H., consulting engineer, Los Angeles, Cal.
Reed, W. M., district engineer for New Mexico.
Ross, D. W., district engineer for Idaho.
Sanders, Wm. H., consulting engineer.
Savage, H. N., consulting engineer.
Swendsen, Geo. L., district engineer, Utah.
Taylor, L. H., district engineer for Nevada.
Walter, Raymond F., constructing engineer for Black Hills region.
Whistler, John T., district engineer for Oregon.
Wisner, Geo. Y., consulting engineer.
Storrs, H. A., electrical expert.

In addition to these there were present at several of the meetings Mr. Gifford Pinchot, chief of the Bureau of Forestry, Department of Agriculture, and four of his assistants; Mr. B. A. Fowler, president of the Salt River Valley Water Users' Association, and many State officials, as well as other prominent persons, some of whom delivered addresses, which are noted or printed in the following pages.

MINUTES OF THE MEETING.

The following brief statement of the principal events of conference gives the names of the principal speakers and the topics discussed:

The engineers of the reclamation service met at the Virginia Hotel on Tuesday, September 15, at 8 a. m., and began the formal discussion of the organization of the service and the methods and results of work already accomplished. The organization of committees and other details were considered. In the matter of the cost and the results of work of supervision and construction a committee was designated consisting of Messrs. George Y. Wisner, A. L. Fellows, and L. H. Taylor.

The relation of the reclamation service to forest preservation was discussed and the importance of close cooperation with the Bureau of Forestry was considered. A committee on forest preservation and utilization was designated consisting of Messrs. Fitch, Gifford Pinchot (chief of the Bureau of Forestry), and Plummer.

As regards the designation of men for specific work and the promotion of those who had shown merit and fitness, there was a general discussion of ways in which to determine the ability and value of each man. The opinion was expressed that it would be very difficult to ascertain the executive ability of any one man through the usual clerical examination, but that by a combination of examination and reports it would be possible to determine merit.

Mr. N. H. Darton, geologist in charge of the western section of the division of hydrology, spoke of the work done on artesian wells and stated that maps had been prepared showing where, in a considerable portion of the West, there was a possibility of obtaining deep or flowing wells.

At 9.30 a. m. the engineers adjourned to attend the meeting of the National Irrigation Congress.

The afternoon session of the engineers was devoted to further discussion of organization and methods of work. Mr. Gifford Pinchot, chief forester of the Department of Agriculture, spoke of the necessity of cooperation between forestry and reclamation work.

The method of making reports from the field was considered at some length, as well as the details of presenting the results of investigation and determining the cost of construction. A committee, consisting of Messrs. Lippincott, Davis, and Fitch, was appointed to recommend committees on the various subjects of interest to the engineers.

On Wednesday, September 16, the engineers of the reclamation service were addressed on the subject of colonization by General Booth Tucker, who brought out a number of points of interest. He claimed that the Salvation Army could furnish colonists, if needed, and could give assurance that these colonists would be competent farmers and desirable citizens.

Senator Reed Smoot, of Utah, spoke to the engineers on the importance of the work and the beneficial results that would result from it in the near future.

Hon. Frank W. Mondell, of Wyoming, extended his greetings to the engineers and expressed his intention of supporting their conclusions in every practicable manner.

The committee on organization, consisting of Messrs. Lippincott, Davis, and Fitch, submitted a list of proposed committees to act as advisers in the technical work and operations of the service.

Mr. Arthur P. Davis described at some length the proposed Salt River reservoir in Arizona. He was followed by Mr. A. L. Fellows, who stated the present condition of operations on the Gunnison tunnel in Colorado. A general discussion of these projects followed.

The engineers then adjourned to attend the meeting of the Irrigation Congress.

At the afternoon session Senator Burton, of Kansas, addressed the engineers on the importance of floods and the interrelation of storage for irrigation with the control of floods.

Mr. J. B. Lippincott described the work in progress in California, particularly that on King and Colorado rivers.

Mr. L. H. Taylor announced that construction work had begun on

the Truckee project, and that surveys, showing the irrigable lands, were rapidly being pushed forward.

Mr. D. W. Ross described three large projects under examination in Idaho, and called attention to the vast extent of irrigable land near Jackson Lake, and the possibility of obtaining an ample water supply for its utilization by storage in that lake.

Mr. Cyrus C. Babb spoke of the work on Milk River, in Montana, and the opportunities of diverting water upon the adjacent lands.

Mr. W. M. Reed described the Hondo reservoir, and also the progress of field work on the Urton Lake project.

At the evening session on Wednesday, September 16, the Hon. Francis G. Newlands, Senator from Nevada, addressed the engineers on the recent Nevada irrigation law, passed largely at his instigation and by means of his active support. This law is unique, in that it compels the most intimate cooperation between the United States reclamation service and the State engineer.

Governor Morrison, of Idaho, commented upon the work of the engineers and the importance of reclamation to the State of Idaho. A number of other public men spoke approvingly, and showed by their words and action the deep interest that is felt by Senators and Representatives and State officers in the progress of the work of the reclamation service.

Mr. B. A. Fowler, president of the Salt River Valley Water Users' Association, described the organization of that body and entered into the various details and the necessary forms of such organization of water users. A general discussion was participated in by the engineers and public men present.

At the morning session, Thursday, September 17, Hon. W. A. Reeder addressed the engineers on the importance of the work and the general interest felt in it.

Mr. John T. Whistler described the surveys in progress in Oregon on Malheur, Umatilla and other rivers. The importance of enacting water laws for the State was discussed, and various features relating to water filings were considered.

Mr. Morris Bien read a decision of the Secretary of the Interior in regard to water rights, which was generally discussed.

The Kansas delegation, headed by Senator Burton and Congressman Reeder, called to pay their respects and inquired concerning the service in Kansas.

The surveys in South Dakota were described by Mr. Charles H. Fitch and Raymond F. Walter, particular reference being made to the Belle Fourche projects.

Mr. George L. Swendsen spoke of the surveys of Utah and Bear lakes now in progress. The chief difficulty in regard to Utah Lake is

the enormous evaporation from it during the irrigating season, amounting to fully 1,000 cubic feet per second.

At the afternoon session on Thursday, September 17, Hon. Malcolm A. Moody, of Oregon, introduced Judge T. G. Hailey, of Pendleton, Oreg., who addressed the engineers on the importance of the reclamation work to Oregon.

Capt. Hiram M. Chittenden, Corps of Engineers, United States Army, spoke briefly regarding the work of surveying and examining the reservoir sites of the arid region.

Mr. John E. Field described the conditions existing on North Platte River, and particularly the reservoir sites on Sweetwater River and below the mouth of this stream.

Hon. Frank W. Mondell, member of Congress from Wyoming, pointed out some of the opportunities for reclamation, and dwelt upon various features of the desert-land law relative to reclamation. A general discussion followed, which was participated in by Messrs. Morris Bien, Arthur P. Davis, and others.

Mr. Jeremiah Ahern described the work on Shoshone River in northern Wyoming, and the topographic methods employed.

Judge F. S. Richards, of Salt Lake City, described briefly the Utah Arid Land Reclamation Commission Fund, which was created by the last legislature, and of which he is the legal adviser.

Mr. Clarence T. Johnston, State engineer of Wyoming, spoke of the facilities offered to the reclamation service by the records of the State engineer's office, and proffered his assistance in making the work successful.

Mr. T. A. Noble dwelt upon the opportunities for reclamation in central Washington, and described in particular the Big Bend Project and the surveys near Okanogan River.

At the evening session Governor George C. Pardee, of California, spoke on the condition of irrigation in his State, and of the importance of cooperation in bringing about a higher development of the country.

On Friday, September 18, the engineers attended the meeting of the Irrigation Congress, addresses being made by Mr. F. H. Newell, chief engineer, Mr. Gifford Pinchot, and others, on the work in reclamation and in forestry.

On Friday afternoon the conference, after completing various details, adjourned.

COMMITTEES APPOINTED.

The following committees were appointed at the conference:

Personnel and Assignments:
- Davis.
- Fitch.
- Savage.

Water Laws:
- Bien.
- Ross.
- Savage.
- Lippincott.

Duty of Water:
- Ross.
- Chandler.
- Code.

Methods of Reconnaissance and Survey:
- Ahern.
- Babb.
- Quinton.

Standard Plans and Specifications:
- Quinton.
- Wisner.
- Savage.

Costs and Results:
- Wisner.
- Fellows.
- Taylor.

Cement and Concrete:
- Duryee.
- Wisner.
- Sanders.

Electric Power Development and Pumping:
- Storrs.
- Sanders.
- McPherson.

Alkali and Drainage:
- Means.
- Sanders.
- Reed.

Forms of Water Users' Associations:
- Bien.
- Savage.
- Davis.

Diamond Drilling:
- Davis.
- Hammond.
- Hamlin.

Forestry:
- Fitch.
- Pinchot.
- Plummer.

HYDROGRAPHIC BRANCH OF THE UNITED STATES GEOLOGICAL SURVEY.

The United States Geological Survey was created by an act of Congress approved March 3, 1879. (Stat. L., vol. 20, p. 394.) From time to time additional duties have been assigned to the Survey by law or by Executive action. A brief summary of the laws is given in the Regulations of the United States Geological Survey, taking effect January 1, 1903.

For administrative purposes the Survey is divided into branches and divisions, as shown on page 8 of its book of Instructions, issued May 1, 1903. The branches are: administrative, publication, geologic, topographic, and hydrographic.

The Geological Survey is one of the bureaus of the Department of the Interior, of which Hon. Ethan Allen Hitchcock is Secretary.[a] The

[a] Hitchcock, Ethan Allen, Secretary of the Interior since February 20, 1899; born at Mobile, Ala., September 19, 1835; attended private schools at Nashville, Tenn.; completed his course at a military academy in New Haven, Conn., 1855; settled in St. Louis and engaged in mercantile business until 1860, when he went to China to enter commission house of Olyphant & Co., of which firm he became a partner in 1866; retired from business in 1872, and spent two years in Europe. Returned to United States in 1874, and was engaged as president of several manufacturing, mining, and railway companies until 1897. United States minister, 1897-98, and ambassador, 1898, to Russia. Home, St. Louis, Mo.

Secretary of the Interior is charged with the supervision of public business relating to patents for inventions; pensions and bounty lands; public lands and surveys; Indians; education; railroads; the Geological Survey; the Hot Springs Reservation, Arkansas; Yellowstone National Park, Wyoming; the Yosemite, Sequoia, and General Grant parks, California; forest reservations; distribution of appropriations for agricultural and mechanical colleges in the States and Territories; custody and distribution of certain public documents; and supervision of certain hospitals and eleemosynary institutions in the District of Columbia. He also exercises certain powers and duties in relation to the Territories of the United States.

The director of the Geological Survey, Hon. Chas. D. Walcott,[a] has charge of the classification of the public lands and examination of the geological structure, mineral resources, and products of the national domain, and the survey of forest reserves, including the preparation of topographic and geologic maps; also the measurement of streams and determination of the water supply of the United States, including the investigation of underground waters and artesian wells; and also the reclamation of arid lands, including the engineering operations to be carried on by the use of the reclamation fund created by act of June 17, 1902, from proceeds of sales of public lands.

The hydrographic branch has charge of all investigations relating to the occurrence of water as a mineral and a source of wealth to the country. It also, through the reclamation service, prepares plans for the construction of reservoirs, canals, and other works for the irrigation of arid lands. The functions of this branch are more fully described in Instructions Relating to the Work of the Geological Survey, May 1, 1903, pages 81–103. The principal divisions, as above stated, are those of hydrography, hydrology, hydro-economics, and the reclamation service.

The division of hydrography measures the surface streams, studies their fluctuations, and determines the quantity of the water supply of the United States. It is divided into two sections, eastern and western, the latter including the thirteen States and three Territories within which work is being done under the reclamation law.

The division of hydrology has charge of investigations relating to the geologic conditions governing water supplies. This division also consists of two sections, eastern and western.

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[a] Walcott, Charles Doolittle, Director U. S. Geological Survey; born at New York Mills, N. Y., March 31, 1850; educated in public schools of Utica, N. Y.; assistant New York State survey, 1876–1879; assistant geologist U. S. Geological Survey, 1879–1888; paleontologist in charge invertebrate paleontology, 1888–1891; chief paleontologist, 1891–1893; geologist in charge geology and paleontology, 1893–94, and since 1894 director U. S. Geological Survey; also secretary Carnegie Institution of Washington since its organization in 1902; member National Academy of Sciences; Fellow American Association for the Advancement of Science, etc.; degrees from Hamilton College, University of Chicago, Johns Hopkins University, and University of Pennsylvania; author of many reports and papers on geology and paleontology.

The hydro-economic division studies the quality of water as related to its uses for industrial or domestic purposes. Attempts are being made to devise rapid methods of field examinations of waters to supplant as far as possible the more expensive and cumbersome analysis.

The hydrographer in charge of the hydrographic branch, Mr. F. H. Newell, is also chief engineer of the reclamation service, and reports to the Director of the United States Geological Survey, who in turn reports to the Secretary of the Interior. The hydrographers and engineers of the hydrographic branch report to the chief engineer, directly or through the district engineers.

The following brief outline gives the form of organization of the hydrographic branch.

ORGANIZATION OF HYDROGRAPHIC BRANCH.

DIVISION OF HYDROGRAPHY.

EASTERN SECTION.

New England.—N. C. Grover, engineer, Orono, Me., hydrographer in charge, assisted by H. K. Barrows, assistant engineer, and F. E. Pressey, engineering aid.

New York.—Robert E. Horton, hydrographer, 75 Arcade, Utica, N. Y., hydrographer in charge, assisted by Clermont C. Covert, assistant engineer, F. H. Tillinghast, engineering aid, and others.

Pennsylvania, etc.—Edwin G. Paul, engineer, Washington, D. C., hydrographer in charge, assisted by Wilbur C. Sawyer, engineering aid.

Southeastern States.—B. M. Hall, resident hydrographer, and M. R. Hall, hydrographer, Temple Court, Atlanta, Ga.; assisted by James M. Giles, engineering aid, and Warren E. Hall, hydrographic aid.

Central States.—Edward Johnson, jr., assistant engineer, 263 Dearborn street, Chicago, Ill., assisted by Frank W. Hanna and L. R. Stockman, assistant engineers.

Texas.—Thomas U. Taylor, resident hydrographer, Austin, Tex.

DIVISION OF HYDROLOGY.

EASTERN SECTION.

M. L. Fuller, geologist in charge; assisted by A. C. Veatch, assistant geologist.

WESTERN SECTION.

N. H. Darton, geologist in charge; assisted by W. C. Mendenhall, geologist, and George B. Richardson, assistant geologist.

DIVISION OF HYDRO-ECONOMICS.

Marshall O. Leighton, hydrographer, Washington, D. C., hydrographer in charge; assisted by R. B. Dole, engineering aid.

RECLAMATION SERVICE.

Created by law of June 17, 1902, authorizing the survey and construction of irrigation works for reclaiming arid lands in Arizona, California, Colorado, Idaho, Kansas, Montana, Nebraska, Nevada, New Mexico, North Dakota, Oklahoma, Oregon, South Dakota, Utah, Washington, and Wyoming from the proceeds arising from the sale and disposal of public lands.

CHIEF ENGINEER.

F. H. Newell, U. S. Geological Survey, Washington, D. C.

EXECUTIVE AND CONSULTING STAFF REPORTING TO THE CHIEF ENGINEER.

Arthur P. Davis, supervising engineer and principal assistant, Livingstone, Ariz., or Chamber of Commerce Building, Denver, Colo.

George Y. Wisner, consulting engineer, 34 West Congress street, Detroit, Mich.

J. B. Lippincott, supervising engineer, 408 Byrne Block, Los Angeles, Cal.

J. H. Quinton, consulting engineer, 1016 West Eighth street, Los Angeles, Cal.

Wm. H. Sanders, consulting engineer, 915 Grand View street, Los Angeles, Cal.

H. N. Savage, consulting engineer, San Diego, Cal.

Charles H. Fitch, supervising engineer, Chamber of Commerce Building, Denver, Colo.

H. A. Storrs, electrical expert, Chamber of Commerce Building, Denver, Colo.

Morris Bien, engineer (on land and legal matters), Washington, D. C.

E. Duryee, cement expert, Phoenix, Ariz.

A. L. Fellows, district engineer, Chamber of Commerce Building, Denver, Colo.

D. W. Ross, district engineer, Boise, Idaho.

L. H. Taylor, district engineer, Reno, Nev.

N. H. Darton, geologist, Washington, D. C.

Charles S. Slichter, consulting engineer, Madison, Wis.

Cyrus C. Babb, district engineer, Malta, Mont.

George B. Hollister, executive officer, Washington, D. C.

T. A. Noble, district engineer, Seattle, Wash.

George L. Swendsen, district engineer, Logan, Utah.

John T. Whistler, district engineer, Pendleton, Oreg.

Gerard H. Matthes, district engineer, Lawton, Okla.

Jeremiah Ahern, engineer for Shoshone district, Cody, Wyo.

John E. Field, engineer for North Platte (Wyo.) district, Chamber of Commerce Building, Denver, Colo.

W. M. Reed, engineer in charge of Pecos surveys, Roswell, N. Mex.

Edwin H. Peery, law clerk, Washington, D. C.

E. C. Murphy, inspector of hydrographic methods, Washington, D. C.

DISTRICT ENGINEERING FORCE.

Arizona.—Arthur P. Davis, supervising engineer in charge, Livingstone or Phoenix, Ariz., and Louis C. Hill, engineer, Livingstone, Ariz.; assisted by W. A. Farish, C. R. Olberg, Archie L. Harris, Willard G. Steward, and R. W. Hawley, assistant engineers; O. T. Reedy, Beauford E. Hayden, Harry T. Holmes, Oliver T. Reedy, Hugh Redmond, and Isaac Voorhees, engineering aids, and others.

California.—J. B. Lippincott, supervising engineer in charge, Room 408, Byrne Block, Los Angeles, and Room 431, Rialto Building, San Francisco, Cal.; Homer Hamlin, engineer on Salinas Valley, etc., 408 Byrne Block, Los Angeles, Cal.; E. T. Perkins, engineer on Colorado River, Yuma, Ariz.; H. E. Green, engineer on Sacra-

mento River, Room 431, Rialto Building, San Francisco, Cal., and S. G. Bennett, engineer on river measurements, 431 Rialto Building, San Francisco, Cal.; assisted by W. B. Clapp, Jacob C. Clausen, and Ralph P. Dillon, assistant engineers, and Linus B. Brainard, Andrew C. Hansen, O. W. Peterson, and Ralph S. Hawley, engineering aids.

Colorado.—A. L. Fellows, engineer in charge, Chamber of Commerce Building, Denver, Colo.; Ira W. McConnell, engineer, Montrose, Colo.; Robert S. Stockton, engineer, Meeker, Colo.; M. C. Hinderlider, engineer, Chamber of Commerce Building, Denver, Colo.; assisted by W. P. Edwards, Edward E. Sands, Andrew Weiss, Fillmore Cogswell, Joseph A. Sargent, and Charles R. Steiner, assistant engineers; E. C. Bebb, topographer; Lavern J. Charles, Oro McDermith, E. R. Childs, Ralph I. Meeker, and Roy H. Bolster, engineering aids, and others.

Idaho.—D. W. Ross, engineer in charge, Boise, Idaho; assisted by John T. Burke, Charles B. Smith, and James G. Camp, assistant engineers; G. H. Hogue, Leon L. Gay, H. S. Williams, William G. Davies, Chester C. Fisher, Robert J. Newell, John L. Savage, Fred Stockton, and N. S. Dils, engineering aids.

Kansas.—Charles H. Fitch, supervising engineer, Chamber of Commerce Building, Denver, Colo.; W. G. Russell, hydrographer, Russell, Kans.

Montana.—Cyrus C. Babb, engineer, Malta, Mont.; assisted by S. B. Robbins, engineer, C. M. Pritchett, E. W. Myers, Alden M. Sprigg, Harry T. Paterson, L. E. Granke, Carlos T. Prall, Josiah L. Rhead, and Goyne Drummond, assistant engineers; Adolph E. Place, Arch. M. Gilbert, and William P. Flynn, engineering aids, and Charles E. Slonaker, observer.

Nebraska.—John E. Field, engineer for North Platte district, assisted by A. J. Parshall, hydrographer; J. D. Stannard, L. V. Branch, Charles G. Carpenter, and L. M. Lawson, assistant engineers; Hugh G. Stokes, Frank C. Magruder, Francis M. Madden, Ray Owen, and W. R. Saxton, engineering aids; J. M. Heizer, field assistant.

Nevada.—L. H. Taylor, engineer in charge, Reno, Nev.; assisted by William E. Swift, consulting engineer; Albert E. Chandler, engineer; Augustus V. Saph, hydrographer; Fred A. Temple, Alfred C. Lootz, William Sargeant, D. W. Hays, W. A. Keddie, Burt E. Forbes, and C. V. Taylor, assistant engineers; L. L. Richard, C. H. Southworth, Edwin P. Arnot, John D. Cameron, Bertram E. Corlett, I. W. Huffaker, and Aug. H. Schadler, engineering aids, and others.

New Mexico.—Arthur P. Davis, supervising engineer, Phoenix, Ariz.; W. M. Reed, engineer in charge of Pecos surveys, Roswell, N. Mex.; assisted by Hurd C. Hurd, assistant engineer; Ernest L. Robinson, Otis L. McIntyre, Earl G. Marsh, Sam G. Porter, Frank S. Dobson, engineering aids, and others.

Also J. A. French, engineer in charge of surveys on the Rio Grande; assisted by George E. Stratton, assistant engineer; Clarence K. Hosford, Williard R. Ewing and Ernest D. Hendricks, engineering aids, and others.

North Dakota.—Charles H. Fitch, supervising engineer, Chamber of Commerce Building, Denver, Colo.; Frank E. Weymouth, assistant engineer in charge; assisted by John N. Kerr and Elwyn F. Chandler, assistant engineers; Frank A. Wilder and others.

Oklahoma.—Gerard H. Matthes, engineer, Lawton, Okla.; assisted by Ferdinand Bonstedt, assistant engineer, on reconaissance of reservoir sites; also Charles E. Gordon, L. M. Holt, Joseph J. Reamer, and Arville C. Redman, assistant engineers; George L. Warner, engineering aid; John E. Fennerty, field assistant, and others.

Oregon.—John T. Whistler, engineer in charge, Pendleton, Oreg.; assisted by W. T. Turner, topographer; Herbert D. Newell, M. D. Williams, and T. B. White, assistant engineers; Edmund I. Davis, Fred W. Huber, John H. Lewis, engineering aids; F. K. Lowry, field assistant, and others.

South Dakota.—Charles H. Fitch, supervising engineer, Chamber of Commerce Building, Denver, Colo.; Raymond F. Walter, engineer, Belle Fourche, S. Dak.

Utah.—George L. Swendsen, engineer, P. O. box S, Salt Lake City, Utah (D. W. Ross, consulting engineer, Boise, Idaho), assisted by William P. Hardesty, William E. Young, and Caleb Tanner, assistant engineers; and Milo H. Brinkley, Thomas H. Humphreys, Henry S. Kleinschmidt, and W. W. McLaughlin, engineering aids. Howard S. Reed, assistant engineer, in charge of stream measurements on Uinta Indian Reservation.

Washington.—T. A. Noble, engineer, 427 Peyton Block, Spokane, Wash., assisted by Christian Anderson, assistant engineer; W. W. Schlecht, assistant topographer; George H. Bliss, assistant engineer; Charles E. Hewitt, Ernest A. Bailey, Calvin Casteel, Charles B. Cox, George F. Harley, and Olaf Laurgaard, engineering aids.

Wyoming.—Jeremiah Ahern; engineer for Shoshone district, Cody, Wyo., assisted by A. H. Horton, assistant engineer; P. M. Churchill, assistant hydrographer; E. S. Ela, assistant topographer; H. R. Evans, Austin C. Downey, Thomas J. Strickler, and C. P. Bowie, engineering aids.

ACCOUNTS AND PROPERTY.

The hydrographers and engineers named in the following list are authorized to certify accounts for approval by the chief engineer, and are responsible to the chief engineer for property purchased in the field.

DIVISION OF HYDROGRAPHY.

Eastern section.—N. C. Grover, R. E. Horton, E. G. Paul, M. R. Hall, E. Johnson, jr.

DIVISION OF HYDROLOGY.

Eastern section.—M. L. Fuller.
Western section.—N. H. Darton.

DIVISION OF HYDRO-ECONOMICS.

M. O. Leighton.

RECLAMATION SERVICE AND WESTERN SECTION OF HYDROGRAPHY.

Arizona.—Arthur P. Davis.
California.—J. B. Lippincott.
Colorado.—A. L. Fellows.
Idaho.—D. W. Ross.
Kansas.—Charles H. Fitch.
Montana.—Cyrus C. Babb.
Nebraska.—J. E. Field.
Nevada.—L. H. Taylor.
New Mexico, Rio Grande district.—A. P. Davis.
New Mexico, Pecos district.—W. M. Reed.
North Dakota.—Charles H. Fitch.
Oklahoma.—Gerard H. Matthes.
Oregon.—John T. Whistler.
South Dakota.—Charles H. Fitch.
Utah.—George L. Swendsen.
Washington.—T. A. Noble.
Wyoming.—Jeremiah Ahern.

REPORT OF THE CONFERENCE.

ADDRESS BY THE CHIEF ENGINEER

[At 8 a. m. on Tuesday, September 15, the meeting was called to order, and the chief engineer made an informal address upon the organization of work of the reclamation service, questions being invited and discussion ensuing on various topics thus brought up. The principal points covered are given in the following pages, various details being amplified, as a result of the discussion.]

PURPOSE OF THE MEETING.

The principal engineers of the reclamation service have been here brought together in this first general meeting for the purpose of becoming better acquainted with each other and with the work of the service, and of exchanging views and information concerning the work in hand and that which is planned for the future. The operations which have been authorized by the reclamation law of June 17, 1902, have expanded rapidly, and in the haste of pushing forward the organization and field work in order to attain important results diverse methods have arisen and important questions have been asked, the answers to which can be given only after more complete knowledge is gained in regard to the work in progress and the men who are executing it.

In a general meeting of the principal members of the reclamation service it is possible to state facts and discuss matters with a thoroughness and understanding which is impossible through formal reports or correspondence. Many details may be talked over here which can not with propriety be brought up elsewhere, and it is desired to have a full and clear understanding of points that may be a source of inquiry or doubt to the men in the field.

The reclamation law requires surveys and examination in thirteen States and three Territories. To carry into effect the objects of th· law, an engineering corps known as the reclamation service has been organized by the Secretary of the Interior as a portion of the hydrographic branch of the United States Geological Survey, in order to get the benefit of the regulations and methods of that Bureau.

The engineers of the reclamation service are, in effect, the advisers of the Secretary of the Interior in the great work of reclaiming the arid lands of the West. They are to obtain facts as to the cost and advisability of various projects and to submit these with recommendations for the construction of certain selected works. The attempt has been made to secure the best engineering experience available and to bring into the subordinate positions in the corps young, well-trained men, competent to carry on field work economically and rapidly. The positions are filled by competitive civil-service examinations. Each district engineer prepares plans and estimates for reclamation work along the rivers under his charge, these being submitted to the consulting engineers for examination and criticism. If approved, the reports are forwarded to the Secretary of the Interior with recommendations for immediate construction.

In making these recommendations to the Secretary, the relative importance of the work is considered, as well as the economy of construction, the cost of reclamation, the character of the land, and all other facts which bear upon the feasibility and probability of bringing about the settlement and thorough cultivation of the reclaimed area. The prime requisite is that prosperous homes may be made upon the irrigated lands. At present the projects are all large and expensive, as most of the smaller reclamation schemes have either been constructed by private enterprise or can be so constructed if left for this purpose.

Actual settlement and cultivation of the reclaimed lands is required by the terms of the law, and there is little, if any, possibility of speculation, since the individual who takes up the land must live on it five years and can acquire complete title only after all payments for water are made. The cost of the works is ultimately refunded, and the money thus returned to the Treasury can be used over again for building other works.

Of the various projects now under consideration, one, in Nevada, is actually under construction. Others are at a point where the specifications and form of contract are nearly ready for submittal to the Secretary. Many projects have been examined and condemned or laid aside for future action. The careful survey and examination necessary to determine the feasibility of these projects necessarily consume time, but all friends of the work are willing and desirous that ample time should be taken to ascertain all the facts and to avoid mistakes. With a well-organized engineering corps actively in the field at work and construction closely following the determination of the essential facts, it will be possible to bring under irrigation, before many years, considerable areas of lands now vacant and useless. The creation from these desert tracts of homes for hundreds of families will make a

wonderful change in the western part of our country, and will create in it a higher standard of living and of citizenship.

The object of this meeting, and, in fact, of all such conferences and all inspections, is to learn how to do things. The duty of an engineer has been stated to be that of accomplishing for one dollar what the ordinary man does for two or more or does not do at all. In private affairs this doing of things rapidly and accurately is difficult in itself, but under governmental regulations the accomplishing of results economically and efficiently requires more skill and forethought, owing to the many necessary checks and requirements of law. The resources of the Government offer wonderful opportunities, but these magnificent facilities for work on a large scale are in part offset by exacting and intricate requirements. In our work we must have not merely good engineers, but men who can and will study the reasons and underlying principles of national law, and who can skillfully guide and direct affairs, meeting all requirements directly and accurately.

It must be borne in mind that the success of the reclamation service depends not only on good engineering work, but also on the constant exercise of sound judgment and of tact in dealing with what may be called the human or social problems. The physical side offers great difficulties to the engineer, but these frequently become less detrimental to ultimate success than the conditions arising from human agencies.

Everything depends on unity and continuity of purpose and of action and the creation of an esprit de corps, or what may be expressed as the holding of a unity of ideals by the entire body of men engaged in the work.

The supervising and district engineers and chiefs of parties should inculcate in the younger men the comprehension that success is dependent largely upon ability to carry out the purpose of the work without minute instructions. Every man must have clearly in mind the object of the work, and be prepared to exercise good judgment in producing the desired end. The ability to accept responsibility and accomplish results is the principal factor to be considered in advancing each man. At the same time there must be a proper observance of rules and regulations in order to conform to essential requirements, but it must be remembered that the strict observance of rules is never an adequate substitute for good judgment.

In this general discussion it is desired to take up first matters relating to the personnel and the relation of the men to one another; next the organization, its strength and limitations; then the appointment of committees and other matters; next the work by committees; then the consideration of the reclamation law, the policy which has been outlined, and the relation to the public in general.

PERSONNEL OF THE SERVICE.

In the work authorized by the reclamation law, opportunities are offered for achieving some of the grandest results conceivable by the human mind, viz, the utilization of the resources of nature and the creation thereby of thousands of prosperous homes throughout an arid region. It is not necessary to dwell upon this matter, but at this time it is sufficient to call attention to the fact that the accomplishment of this great work necessitates the creation and organization of a thoroughly well-equipped body of men. The Secretary has thought fit to intrust this work to the Director of the Geological Survey, and he in turn has authorized the formation of a corps of engineers and experts as one of the parts of the hydrographic branch of that Survey.

The rules and precedents governing the Geological Survey are, it is believed, among the best of those of any Government bureau. There is an elasticity about the organization which enables individual development, and there is a constant attempt made to conduct affairs on purely business principles, and yet with scientific care and accuracy. It is without doubt an honor to be a member of the Geological Survey, and it is hoped that the high standard that has prevailed in that corps of scientific workers will be maintained and advanced in the reclamation service.

Every effort should be made to attain the highest possible efficiency in every branch of the work. The men who come into the regular force of the reclamation service do so through competitive civil-service examination. It must not be supposed, however, that a man who passes the examination and is appointed has met the only requirement. This is but the beginning; and no man should be retained in the service who does not keep up to the standard set. It has been found desirable, from time to time, to intimate to various men who have civil-service appointments that they are not performing their duties in the best manner possible and that unless marked improvement is seen in their work their resignations will be accepted. This hint has generally been sufficient.

It should be borne in mind at all times that we can not be satisfied with men who are simply "good enough." If a man is mediocre or takes only a perfunctory interest in his work, this fact should be clearly stated and he should be induced to seek employment elsewhere. This applies particularly to the younger men who are just entering the service. It is sometimes the very best kindness to them, as well as a duty to the organization, to show them that employment in this service means efficiency and devotion to work.

During the past year or two the papers of several hundred men have passed through my hands, and I have talked with many of these

or have had correspondence with them. A great many people endeavor to enter the Government service, and some of them have extremely curious notions regarding the work and what is required of them. It is difficult to convince some of these people that the Government service demands that the best business principles must be followed, and that every effort is being made to find out the ability of each man and give him recognition accordingly. Indorsements or importunities from any source other than those of a strictly business character are not needed, and may be detrimental, especially if employment is sought on political grounds. No considerations of sentiment, religion, or politics should have any weight, but each man should be judged solely by his character, ability, and physique.

There is a special provision in section 10 of the civil-service law (approved January 16, 1883), regarding recommendations by members of Congress. This is frequently overlooked by persons who have the mistaken idea that they may be appointed or advanced by the use of influence from this direction. It reads as follows:

> That no recommendation of any person who shall apply for office or place under the provisions of this act which may be given by any Senator or Member of the House of Representatives, except as to the character or residence of the applicant, shall be received or considered by any person concerned in making any examination or appointment under this act.

There are also men who desire a position on the reclamation service for the advertising it may bring them and with the hope of obtaining through it occasional outside employment as specialists. It has been found wise to require that each man connected with the Geological Survey or its branches shall devote all of his energies to the work for which he is engaged and to abstain from any other occupation for which he will receive remuneration. In short, his entire time and energies should be devoted to advancing the work on which he is engaged.

Great care must be taken in the case of new men to see that they are physically able to endure the work. A number of well-educated and experienced men have fallen out on account of lack of physical strength. In one or two instances men have been transferred without specific inquiry into this matter, and considerable expense and annoyance to the service and to the friends of these men have resulted, as they have developed sickness when far away from medical treatment or means of transportation.

The opinion seems to be prevalent that if a sick man or one partly well can only get employment in a Government party he will have opportunity for rest and recovery. This is true particularly as to persons afflicted with consumption, many of whom have sought in every possible way to get employment in the arid West. In a few instances such men, apparently in good health, have become attached

to field parties, but the hardships have been too severe, and they have succumbed. While we have pity for such unfortunate cases, we can not encumber the field work by employing these men, nor can we afford to run risks of injury to them or to others.

Great care must also be exercised concerning men of another class, who are trying to get into the reclamation service. These are young college students or graduates, who merely desire summer occupation or an outing. A number of such men have been obtained through civil-service certification, and after they had been transported to different points they have stated that they did not intend to continue in the work and have gone elsewhere. Attempt is made to ascertain in advance whether each new man intends to remain indefinitely with the service; but if men who are restless or who change their minds are employed, it is desirable to get rid of them as soon as possible in order not to waste money on their education. Much of the investigation being carried on has high educational value, and the men who are thus trained should be retained as far as possible for the future work of the service.

Occasionally a man has been found to be good, but not worth the salary paid. In such a case recommendation should be made for its reduction to the amount which he is actually worth, and opportunity should be given for him to work up if he shows capacity and energy. On the other hand, if a man receiving a low salary shows unmistakable evidence of skill and administrative ability his promotion should be recommended.

Attempts should be made to impress upon all of the men in the service not only the necessity of good work, economically conducted, but also the importance of keeping up appearances, both personally and as to the camp. A field suit of khaki can be obtained at a small price, and the men should be encouraged to wear these suits whenever practicable and to avoid a slouchy, ragged, or untidy appearance. Similarly, the camps should be arranged in an orderly manner, the tents placed symmetrically, and the spaces between the tents kept in a neat condition. It requires no particular outlay of time and money to preserve good appearances, and these have a reflex action upon the work. Accuracy, economy, and neatness are usually coordinate.

It must be remembered that the general public judge largely from outward appearances, and that each man is, to a certain extent, responsible for the others and carries the reputation of the entire service in his hands. Many things which he might properly do were he working for himself he can not do as a member of an engineering corps; he must at all times remember that he is a part of an organization of which the whole may be judged by that part.

A phrase has been suggested here which is peculiarly applicable—that is, the necessity of good "team play." Each man is one of the

team, and the successful play can be made only by every man performing quickly and accurately his especial part as the emergency arises. No one is sufficiently independent of the others to carry on the play alone. We must have not only the very best men, but these must be capable of working together coordinately as parts of the whole. Many men have been found who are good in themselves, but who can not safely form an integral part of a great organization, since they do not have the fundamental ideas of good "team play." Each of these must be the captain or nothing, and their talents are wasted, as far as the reclamation service is concerned, by their failure to cooperate and assist others.

All of the work in hand, especially that of reconnaissance and preliminary surveys, is, to a certain extent, full of emergencies. Countries partly explored or almost unknown are examined and unexpected contingencies are constantly arising. Weather conditions play a large part, and the seasons are frequently such that certain classes of work are impossible. Opportunities must be seized and personal convenience must not be regarded. When floods occur they must be measured, and when the sky is clear signals must be observed even at personal sacrifice. For this reason heat or cold and times or hours of work can not be considered, and if any man in the regular service begins to complain of irregularities of hours of employment do not hesitate to recommend him for private occupation.

When there is a particularly good man in any part of the country, it is, of course, the desire of the party or district chief to keep him there, and it is often more convenient to retain in one field the services of such a man; but, on the other hand, some consideration must be given the man himself in order to develop and build him up. For this reason it is desirable to recommend the occasional shifting of men from one district to another in order to afford them opportunities for wider observation and for the development of judgment in engineering work. Do not recommend for transfer only the men whom you may wish to get rid of, but also bear in mind the desirability of giving each man broad experience, as far as the work will permit.

OFFICERS OR REGULAR APPOINTEES.

There are two classes of men employed on the reclamation work—those who are regularly in the service and those who are employed temporarily, from time to time, as occasion may require. Those who are regularly employed hold an appointment from the Secretary of the Interior, and have been selected, after competitive examination, from lists furnished by the Civil Service Commission. All of the men who have been thus regularly appointed are officially designated on the Government register as "temporary," this being the designation of

those persons whose office is not specifically created by act of Congress. No positions in the reclamation service were thus specifically created, and hence all of the men are officially in the category of temporary, though regular, employees. This designation need not alarm any one, as the greater portion of the force of the Geological Survey consists of "temporary" appointees, some of whom have been twenty years or more in the service of the Government and intend to devote the remainder of their lives to the work. A few persons in the Geological Survey are not "temporary," such, for example, as the director, chief clerk, chief disbursing clerk, a few messengers, etc., but most of the geologists, topographers, hydrographers, etc., are designated as "temporary," even though their terms of service may extend over the greater part of a generation.

It is, in one way, an advantage to be a temporary appointee, since the compensation of such an employee is not fixed rigidly by law, and in his case it is possible to pursue the methods of ordinary commercial life—that is, his compensation may be increased or reduced at the will of the Secretary and arranged in accordance with the ability of the employee and the character of the duties performed by him.

The regular employees of the reclamation service are classified, in accordance with duties and compensation, into various grades, ranging from the aids, at $60 per month or even less, up to the consulting engineers and chief. The aids, as a rule, are young men, fresh from college or technical school, who are supposed to have had a thorough education in civil, hydraulic, electrical, or mechanical engineering. Their compensation is usually $60 or $75 a month for the first six months, or until ability has been shown in the field. As a rule, attempt is made to procure young men at a nominal compensation, in order to give them a trial and determine what they are worth.

After service in the field the aids are promoted from $720 to $900 or $1,000 per annum, and after one or two years of service promotion to the position of assistant engineer may be considered.

When this grade is attained the appointees are considered as regular officers of the reclamation service, while all below the class of assistant engineer—that is, below class 1 of the civil service—are habitually spoken of as occasional employees. The regular officers are furnished with an identification card for convenience in transacting business.

The assistant engineers are men who have had a certain amount of experience in the field, and who have been found competent to handle small field parties and suggest or initiate minor branches of the work. As the organization now stands, a considerable number of the assistant engineers are men who have come in from private practice and who have had considerable experience in surveying and constructing irrigating ditches. Not all of the men are graduates of colleges or tech-

nical schools, for the various grades of assistant engineer have been sometimes filled by the promotion of young men who have been educated in the service as engineering aids. Assistant engineers are paid $1,200, $1,400, and $1,600 per annum.

Officers designated as engineers receive $1,800 or more per annum. They are required to have sufficient experience and ability to carry on independent work and to direct the movements of several field parties. Advancement is made, as ability is demonstrated, to $2,000, $2,200, or $2,400, and ultimately, as the value of the judgment of the men is demonstrated, they may become consulting engineers at $3,000 or more.

Entrance into the regular service is gained as above stated, through competitive civil-service examination, and by selection, under customary rules, from the lists submitted by the Civil Service Commission. It is proposed each year to have two examinations one of these will be held in the spring (probably in April) to secure young men who are about to graduate from college, and the other will be held in the fall (probably in October) to give an opportunity to more experienced men. In the latter examination experience counts largely and book learning plays a less essential part.

ORGANIZATION.

Next in importance to the character of the men themselves is the general plan of organization, and to this much time and thought has been devoted during the past year. With the rapid growth there has been and will continue to be a gradual shifting, but in a general way it may be said that the organization of the reclamation service is now fairly well outlined.

DISTRICT ENGINEERS.

The grouping together of the various grades of aids, assistants, engineers, and consulting engineers, is based largely on geographic divisions. In each of the thirteen States and three Territories named in the reclamation law, there are one or more district engineers, the district being the basis of the organization and embracing an important river basin. The district, for convenience, is designated by the name of the State rather than by that of the river, provided there is only one district recognized in a State.

The district engineer has charge of all of the work in his district, and matters are referred to him for report. He corresponds directly with the chief engineer, prepares quarterly estimates of expenditures, approves all accounts, and is responsible for all Government property. Each district may be regarded as the unit in the field organization.

There are assigned to the district engineer other engineers to take charge of various parts of the work, and under these, in turn, are assistant engineers and aids to execute the details. The assistant

engineer, as a rule, has charge of a small party, and acts as principal instrument man. The aids act as recorders, rodmen, or occasionally use level and transit, acquiring experience which will enable recommendations to be made for their future promotion.

The district engineer receives the reports of the assistant engineers, arranges and digests the information, and elaborates the results into a definite statement, with recommendations concerning the feasibility of some one reclamation project or another. At the same time he directs the preliminary surveys and examinations that are conducted in various parts of his districts, and keeps all matters moving forward along the lines approved by the chief engineer.

When a district engineer has prepared for presentation his detailed report, with plans and estimates, on any particular reclamation project, it is transmitted to the chief engineer and by him referred to a board of consulting engineers convened for the purpose and instructed to ascertain all the facts, to verify the conclusions, and to approve or disapprove the recommendations.

There may be distinguished three stages in the development of any great project of reclamation: First, the exploratory and location surveys; second, the designing; and third, the execution.

The first stage, that of exploration and location, is carried on by the district engineer and his assistants, with aid from a supervising engineer. His conclusions are then passed upon by a board of engineers, of whom the supervising engineer is one of the members.

The second stage, that of designing, is next taken up by the district engineer with the assistance of specialists in the particular line of construction. The results are again passed upon by the same or another board of engineers, and if the results are approved by the Secretary of the Interior the project passes to the third stage, that of construction.

When a project reaches the construction stage, experienced constructing engineers are detailed to the work, these men being taken as needed and assigned to the particular work in hand to stay with it continuously and supervise and inspect every detail of construction. In short, in all stages of progress the attempt is made to provide men specially fitted by education and experience to carry on the work in hand. At the same time the number of men is kept at the minimum and the force increased only where needed.

With this form of organization, it is expected that the experienced men in the field will initiate ideas and make suggestions and recommendations, to be passed upon by the chief engineer and boards of consulting engineers. These ideas may be modified or expanded, but the success of any man rests largely upon his intelligence in grasping the general situation, and his ability to execute readily and economically

the conceptions which, originating largely with himself or his assistants, are approved before being put into effect.

SUPERVISING ENGINEERS.

The supervising engineers are in effect deputies of the chief engineer and possess all requisite authority to execute the work which may be delegated to them. As it is impracticable for any one man to supervise all of the operations of planning and construction, the territory of the supervising engineers is designated in general terms and is being in each case gradually narrowed, additional supervising engineers being designated from time to time as the work progresses. The supervising engineers are men of experience in engineering affairs and in executive work, and they may serve as consulting engineers, but it is assumed that the greater part of their time will be used in the executive work pertaining to the conduct of affairs in the various districts under their charge.

CONSULTING ENGINEERS.

The consulting engineers are men employed continuously to give advice and suggestions concerning various details of the engineering work. They have few, if any, executive functions, but upon them is placed the responsibility of determining engineering details and policies. They are not limited geographically in their operations, but their sphere of activity is rather in specialization of operations. That is to say, the consulting engineer of most experience in cement work is called into consultation in any part of the country where important questions are raised concerning the use of cement in large quantities.

The consulting engineers are generally organized into boards, as the problems presented in any one locality may fall into various departments of engineering. They travel widely, and meet according to prearranged programme, bringing together such of the men as are believed to be best qualified to pass upon the broad problems met in each project. The supervising and district engineers are usually called in consultation wherever practicable, but the main features are left to the decision of the engineers employed in a consulting capacity.

PLACING OF RESPONSIBILITY.

In the organization of the reclamation service great care has been taken to preserve and place direct responsibility upon each individual, so that he appreciates his relation to the work and is stimulated by the fact that he will receive recognition or disapproval according to his actions or judgment.

In widely diverse and broadly scattered work of this kind a carefully graded organization is needed to insure the complete responsibility of

individuals and to have the burden bear upon each individual precisely in proportion to his experience and opportunities.

Primarily, the Secretary of the Interior, as required by law, is the responsible officer. He in turn holds responsible the Director of the Geological Survey, and he in turn the chief engineer of the reclamation service. Under the direction of the latter are the supervising and consulting engineers, each of whom, acting individually or in boards, submits carefully prepared written statements and recommendations covering each important point. Next come the district engineers, who have the direct and immediate responsibility for all work in their districts, and under these are the designing and constructing engineers, the inspectors, instrument men, etc., each with his specified line of work.

The two principal ends aimed at by the organization of the reclamation engineers as outlined are, first, the fixing of responsibility definitely upon those engineers to whom the execution of a project is assigned; and, second, the securing of such elasticity in the organization that the best talent possessed by the corps of engineers may easily be applied to the special needs of the service.

In order to attain these ends the execution of each project is placed in the hands of a limited number of engineers, preferably three—viz, a resident engineer, a district engineer, and a supervising, directing, or consulting engineer, whose duties are as follows:

(*a*) The resident engineer is directly in charge of the construction work, and reports to the district engineer.

(*b*) The district engineer has general supervision of the execution of the project, and reports to the chief engineer.

(*c*) The supervising, directing, or consulting engineer is definitely attached to the given project, being selected and assigned to that duty by the chief engineer, and he reports to the chief engineer only in conjunction with the district and resident engineers.

These three engineers constitute what may be termed the "project board" for a particular project. They are individually and jointly responsible for the successful carrying out of the approved plans covering a project, and for all expenditures incurred in connection with the execution of the project.

The resident engineer devotes himself exclusively to a particular project; the district engineer divides his attention among all the projects, whether contemplated or approved, lying within his district; the supervising engineer divides his attention among all the projects, regardless of districts to which he is assigned.

The district engineer keeps in close touch with the details of the work being done by the resident engineer, so that practically nothing will be done without the joint knowledge and approval of these two members of the project board.

All reports, estimates, and general statements covering expenditures are made out or approved in the office of the district engineer for transmittal to headquarters.

The supervising engineer gives sufficient study to the project to keep himself adequately posted regarding the progress of the work, so that he considers himself equally responsible with the other members of the project board for all work and expenditures involved in the execution of the project. To this end he visits the project as often as he or the district engineer deems necessary, and receives from the district engineer such reports of the work being done or contemplated as will enable him to prevent the carrying out of any plans or the making of any expenditures of which he does not approve and for which he is not willing to be held responsible.

The supervising engineer, at the request of the chief engineer or the district engineer, considers and definitely reports any feature of the project upon which a final decision is required.

The second object aimed at—viz, elasticity and adaptability as regards the character and qualifications of the engineers of the service in relation to the varying demands of the work—is secured by means of the following arrangement:

All the supervising engineers are members of and constitute the board of supervising engineers. At least one supervising engineer is usually present in the Washington office, and at the direction of the chief engineer fills the position of acting or deputy chief engineer. Whether the chief engineer is present or absent, the supervising engineer, who is designated as acting chief engineer for the time being, dispatches all the business of that office within the limits of his authority and ability.

BOARDS AND COMMITTEES.

While the reclamation service, for effective administration, is and must be a one-man organization, in the sense that one man is responsible at each stage of the proceedings, yet at the same time, for consulting and advisory purposes, it is organized into a number of committees—that is to say, in the execution of the work, the chief engineer, or the district engineer, or the engineer in the field, or the assistant, etc., is directly responsible for certain details, and the responsibility is rigidly enforced—yet in matters of opinion the advice of a number of men is sought. For this purpose committees are constituted to advise the chief engineer. These committees consist of men who are familiar with certain details, and whose knowledge is needed in making plans for the future. There are many of these committees; some of them constituted for temporary purposes and then discharged; others practically continuous, the personnel changing

gradually. The chief engineer is ex officio a member of all committees, even though not mentioned among the names. A list of committees appointed at the Ogden meeting is given on page 14.

An arbitrary distinction is made between boards and committees in that the term "board" is used to include occasional organization of engineers in the field for consideration of specific projects for construction and to take up special points of engineering practice. Under the term "committee" is included the office organizations or groups of men appointed at engineering conferences to consider methods of transaction of business, coordination of the various parts of the reclamation service, legal or physical questions which pertain to the operations of the service, and various matters of this kind which relate more directly to administrative operations.

All actions of boards or committees should be unanimous, if possible, and a carefully prepared report should be signed by all members, each becoming individually responsible for the accuracy of the statements or for the correctness of the judgment. These reports should be addressed to the chief engineer, and should be as explicit and concise as possible.

COMMITTEE ON PERSONNEL.

One of the most important committees is that on personnel. This is constituted of persons who are best acquainted with the men and their work, and who visit the various field parties and form an opinion of the operations. This committee makes recommendations, from time to time, regarding the advancement or dropping of certain men or their assignment to various classes of work. It is expected that this committee will make such recommendations to the chief engineer in May and November, and that these recommendations will apply to the entire list of regular employees. Each district engineer, and in some cases a chief of field party, will be expected to state to the committee confidentially his opinion as to the relative ability of the men working under his direction. Action by the committee is to be based wholly upon the ability of each man and the results accomplished by him in his line of work.

Any importunities or attempts to influence the judgment of the committee or of the chief will count against the men making them. There is, unfortunately, an opinion prevalent in certain quarters that in Government work it is necessary to secure the endorsement and continued support of some politician who will act as a sort of patron and continually urge the officers of the Department to advance his protégé. This, as far as the Geological Survey and its branches are concerned, is wholly untrue, and there is nothing that an employee can do that will lower him more in the estimation of his fellow members and of the responsible officers of the Bureau than to try to bring such

influence to bear in his behalf. An attempt made by any man in the employ of this Bureau of the Government to procure advancement in other ways than by demonstrating his ability to accomplish his designated work is regarded as a sign of weakness and incapacity. Such a man is at least under justifiable suspicion.

The committee on personnel should consider, as part of a man's qualifications for advancement, not merely the results which he has accomplished, but his fitness for larger duties. This should include his tact and ability to get along well with men and to manage efficiently the party or parties that may be placed in his charge. An effort is made to keep careful account of expenses incurred in various lines of work, and a man's record for economy is an essential matter for consideration.

As regards those younger members of the service who have not yet had opportunity to show their ability in large matters, consideration should be had of the neatness and accuracy of their work and of the impression which they make. It is not desirable to advance a man who is habitually careless in his personal appearance and address, as it is presumable that he is weak in other matters.

There are also suitable committees to take up various other matters, such as the consideration of the water laws of the various States, the duty of water, methods of reconnaissance and survey, standard plans and specifications, costs and results of survey and construction, tests for cement and use of concrete, electrical power and its development and use in pumping, alkaline drainage, the forms of association of water users, methods and equipment of diamond drilling, the protection of drainage areas by forestry, etc. All of these and other important points should be considered by qualified men, and the results should be discussed by all of the engineers of the service.

CIVIL-SERVICE REQUIREMENTS.

The operations of the reclamation service, as well as of the Geological Survey in general, must be carried on in conformity with laws governing the civil service and the rules and regulations of the Civil Service Commission. These laws and rules have been drawn mainly with reference to the systematic, methodical conduct of Government business at Washington and in the larger offices throughout the country. They designate how persons shall be employed, and cover in a broad way the usual requirements of departmental duties. In the Geological Survey, however, and particularly in the reclamation service, where work is being initiated and carried on in unsettled or almost unexplored regions, the regulations that cover ordinary office routine are not always applicable. The civil-service laws do not distinctly recognize such unusual conditions, but provide for exigen-

cies and give to the Civil Service Commission a certain amount of discretion in enforcing the letter of the law. The Commission, recognizing the unusual nature of the duties of the Geological Survey and its branches, has been liberal in its requirements and has done everything possible to facilitate the work of the Bureau. This liberality on the part of the Commission must be respected, and every attempt should be made by men in the field to live up to the letter and spirit of the law and preserve the confidence of the Commission that the law will be observed in all particulars.

All persons regularly employed in the service of the Government are grouped into grades and classes. In any given grade are included all persons performing certain kinds of duty, irrespective of their compensation; that is to say, the engineer grade includes all men who are doing engineering work, whether they are paid $1,000 or $2,000 a year or more. Promotion within a grade can take place without special requirements of examination, but men can not be transferred from one grade to another without some form of examination or certification as to ability to perform the duties of the new grade.

Within each grade persons are divided into classes, according to the amount of compensation received. The term "grade," in connection with employees or positions, refers to a group of employees or positions in the classified service arranged upon the basis of duties performed, without regard to salaries received, as messenger grade, clerk grade, engineer grade; while the term "class" refers to a group of employees or positions in any grade arranged upon the basis of salaries received, in pursuance of section 163 of the Revised Statutes and of section 6 of the civil-service act. A person regularly serving in any class in a given grade is not required to undergo examination for promotion to any other class in the same grade.

Classes 1, 2, 3, and 4 were created in 1853, and since then from time to time other classes have been added above and below these, until at present the arrangement is as follows:

Class A. All persons receiving an annual salary of less than $720, or a compensation at the rate of less than $720 per annum.

Class B. All persons receiving an annual salary of $720 or more, or a compensation at the rate of $720 or more, but less than $840 per annum.

Class C. All persons receiving an annual salary of $840 or more, or a compensation at the rate of $840 or more, but less than $900 per annum.

Class D. All persons receiving an annual salary of $900 or more, or a compensation at the rate of $900 or more, but less than $1,000 per annum.

Class E. All persons receiving an annual salary of $1,000 or more, or a compensation at the rate of $1,000 or more, but less than $1,200 per annum.

Class 1. All persons receiving an annual salary of $1,200 or more, or a compensation at the rate of $1,200 or more, but less than $1,400 per annum.

Class 2. All persons receiving an annual salary of $1,400 or more, or a compensation at the rate of $1,400 or more, but less than $1,600 per annum.

Class 3. All persons receiving an annual salary of $1,600 or more, or a compensation at the rate of $1,600 or more, but less than $1,800 per annum.

Class 4. All persons receiving an annual salary of $1,800 or more, or a compensation at the rate of $1,800 or more, but less than $2,000 per annum.

Class 5. All persons receiving an annual salary of $2,000 or more, or a compensation at the rate of $2,000 or more, but less than $2,500 per annum.

Class 6. All persons receiving an annual salary of $2,500 or more, or a compensation at the rate of $2,500 or more per annum.

This classification is recognized by the Department, and classes 1, 2, 3, and 4 are recognized by the Revised Statutes.

The new code of rules of the Civil Service Commission, brought up to September 1, 1903, should be carefully read and frequently consulted by members of the reclamation service in order that its provisions may be observed. Each member should also obtain the annual reports of the Commission by application to the Civil Service Commission, Washington, D. C., in order to keep in touch with the changes in the rules or the application of rules to specific cases.

In attempting to apply these rules to the work of the reclamation service it is seen at once that a certain amount of interpretation of general statements and a certain degree of executive discretion is necessary. As a rule the work of the reclamation service is primarily that of exploring or ascertaining facts in advance of a knowledge of the limiting conditions. Field parties must be equipped and organized in new regions and care must be taken so to arrange and proportion these parties that work will be carried on efficiently and economically. It is obviously not practicable to assemble a considerable number of men, instruments, animals, and supplies at remote points, sufficient to cover all contingencies, since the cost will be practically prohibitive. On the contrary a few well-equipped men must be employed at first and additional assistants and supplies sent to these men quickly and economically, wherever they happen to be, and the chief of party must be in a position to procure necessary assistants without delay. In short, a chief of field party operating 50 or 100 miles from the nearest telegraph station can not be expected to send to Washington for men or material to supply each want as it arises. To a field party costing $40 to $60 or more a day, any delay, even of an hour, means considerable loss, and the exigencies are such that even with all possible foresight and oversight, freedom must be left to the chief of party to obtain and procure needed assistants.

For this reason departmental practice and the Civil Service Commission have permitted and encouraged the exercise of judgment in equipping and maintaining field parties. It is usually very difficult to procure competent men, and it is not possible to wait for certification in the midst of a short field season. For example, in the case of cooks, while for an Indian school or Government hospital a civil-service

examination or registration might be desirable since it is extremely difficult to get and retain any competent man in this position, yet for a field party examination would obviously be absurd.

Progressing from such extreme cases upward, there comes a grade, however, where it is possible to foresee and expect continuous employment, and these men, who are responsible assistants, should be selected through the civil service and provided in advance of work. Each chief of a field party is expected to ask in advance for men or to take such men as may be assigned to him by the district engineer or chief engineer, and in organizing the work to employ such other men in the field as may be needed from time to time. Men regularly employed season after season are, as required by law, in the classified civil service. The other men, taken for occasional or job work or needed as exigencies occur, are employed as field assistants and must be so designated on the vouchers, the word "field" being used to distinguish the men from others of similar grade in the classified service. For example, if an assistant engineer becomes sick, or his place must be temporarily filled, or if an additional assistant engineer is required immediately, the person thus employed should be designated as "field assistant" or "field engineer," not as assistant engineer, as this position is in the classified service. The employment of such men in the field is in the nature of job work, for a specific purpose, and is not regular employment in positions existing or created. The men are employed not to fill a definite position, but to perform certain acts that are necessary to carry into effect the provisions of law concerning work authorized by Congress.

When, however, it becomes apparent that such employment may be permanent in character, then application should be made for certification of suitable eligibles, and the requirements of the case should be stated plainly and explicitly. The report should contain the following facts: The name, age, occupation, and experience of the person temporarily employed; the character of work for which he is employed; the probable term of service; and if the work will probably be of a continuous nature a request should be made for the appointment of a qualified person to fill this place, all of the facts in the case and the needs of the work being plainly stated. This report should also state the rate of compensation, the fact whether subsistence is furnished in camp, and whether transportation to or from the place of employment is to be included, as well as all other matters that may serve to aid in the selection of a proper man to be sent to the designated point.

Allowance must be made for the length of time required to request such appointment through the ordinary channels; that is to say, a chief of party in the field, away from ordinary lines of travel, should, when necessity arises, employ the nearest available man suitable for the purpose, and then promptly make statement of the fact, and, if

necessary, request the designation of a person to fill the place. This request should come to the office of the chief engineer for indorsement; it is then sent to the executive division of the Geological Survey, from there to the Secretary of the Interior and to the appointment division of his office, and finally to the Civil Service Commission. Here the records will be searched, names certified, and papers sent through the same channel and forwarded, if necessary, to men in the field. This will require, from the time inquiry is started, a minimum of at least two weeks, and may require a month or six weeks. When the names are before the chief engineer, or the engineer in the field, he must take up correspondence with the persons certified to ascertain whether they are then available and whether they will accept work at the designated point and position. If, for example, a man employed on engineering work in Massachusetts is certified for work at some remote point in Arizona he will generally desire to correspond about the matter, and may require to give his employer a month's notice—thus a month may be consumed in correspondence and in getting ready to start.

Experience has shown that the greater part of the field season is frequently consumed in getting the right men through the ordinary routine, and great patience is required, as well as strict attention to all of the details and correspondence. This condition must be borne in mind in judging of the exigency of any particular case, and good judgment must be used at all stages in endeavoring to comply with the law and at the same time in pushing forward the work economically and rapidly.

Difficulty or delay in procuring the certification of proper persons will not be accepted as an excuse for delays in the field, as everywhere the field party and district engineer must bear in mind the fact that the duty is first to perform the work, and, secondarily to this, to carry it on in compliance with official rules and regulations. Results must be had economically and effectively, and no excuse of so-called "red tape" can be considered.

Taking up in detail the grades of men employed in the field, and beginning with river observers, it is to be remembered that these are not included within the rules of the civil service, even though employed year after year indefinitely. These men are paid usually from $3 to $5 a month and in some cases a higher rate, as where they are located at remote, inaccessible points. It would, for example, be obviously impossible to obtain a certification for an observer to live at a certain designated point or ranch, many miles from other habitations, at such a salary. In such cases there is probably only one man—the occupant of the ranch—who could be induced to make river observations.

Cooks, drivers, packers, and laborers in general do not come under the cognizance of the civil-service rules, since these men are employed

only for certain specified work for a few days or a few months, and are hired and dropped as the exigency requires, or, more frequently, as they can be obtained, as this class of labor in the West is very uncertain and difficult to obtain.

Various technical experts, mechanics or mechanical engineers, electricians, cement experts, etc., must be employed from time to time to assist in planning or construction. When there is reason to believe that the services of such men will be required, they should be employed under the operations of the rules. When, however, an exigency exists, the district engineer will at once make use of the service of the best available man and report the fact as above indicated.

Young men to act as rodmen, station assistants, and to perform functions more or less technical in character, fall about the border line of the civil-service rules. To properly organize a field party several of these may be required, and as far as possible a register should be established of these men, or they should be obtained in advance from among those regularly employed. If, however, it is found that additional help is required, young men may, under existing custom, be employed at the point of outfitting and retained throughout the field season, or until registered or classified men properly qualified may be had.

Coming to the higher grades of assistant engineer, engineer, etc., these men should all be within the classified civil service. In case, however, of need of an expert man, or failure of an assistant regularly employed, the chief of the party or division engineer must substitute at once a qualified person without loss of time. The facts of the case should be reported as above designated to the chief engineer, and the place, if permanent, filled through transfer or certification from the Civil Service Commission.

Summing up the condition as regards laws and regulations relating to the civil service, the employees of the reclamation service may be considered in four schedules, as follows:

1. Laborers not in the classified civil service and not appointed.
2. Occasional or job employees not in the classified civil service and to whom appointments are not given.
3. Surveyors and mechanics who may be taken from a registered list prepared, without educational tests, by a civil-service board.
4. Engineers, assistants, and aids, appointed after competitive educational examination and duly certified by the Civil Service Commission.

It is not practicable to classify these men by compensation, for, in the parts of the country where the work of the reclamation service is being carried on, the compensation paid must vary with the conditions and needs of men; that is to say, men who under ordinary circumstances in or near a city should be paid $2 a day may receive $4 per day in remote districts.

Laborers.—Under this heading are included all unskilled laborers and those employed as axmen, chainmen, boatmen, cooks, derrick or drill hands, drivers, packers, and general assistants about a camp or construction outfit.

Occasional employees.—Under this classification are included all persons above unskilled laborers in rank, who are not appointed, but who are occasionally employed, and whose occupation is such that it is not possible to establish a register for them, either because of the character of the work or because it is only temporary and of the nature of a job. Under this head there are also included temporary assistants, men taken in an emergency to fill a need when it is impracticable to procure competent assistants from the register or by certification from the Civil Service Commission. Various technical experts are also included, their work being of the nature of a job to be performed.

Surveyors or mechanics.—Under this designation are included all skilled assistants whose work is of such character that an educational test is not desirable or necessary, and who may be employed in cases where skill is a prime essential. In each district a register of capable men should be established at the earliest practicable date, and selections made from this register in accordance with civil-service rules. The register is to be established by a local board, acting as the agent of the Civil Service Commission and reporting directly to that Commission. In this schedule are included inspectors, overseers or foremen, master laborers, master carpenters, recorders, transitmen, levelmen, rodmen, timekeepers, cement testers, gage readers or observers receiving over $50 a month, telephone linemen, and similar employees.

Engineers.—In this schedule are included all engineers, assistant engineers, engineering aids, and all employees filling similar technical positions, constituting what is known as the regular force of the reclamation service. All of these positions are filled by competitive educational examination, and, as far as practicable, assignments to responsible positions are made from the various classes of engineering aids and assistants. The civil service rules are particularly applicable to all persons included in this schedule, and great care must be taken to observe all of the requirements of law or of regulations pertaining to this regular force of employees.

Local boards.—Under authority of the Civil Service Commission a local board of examiners may be designated in each of the reclamation districts. This board consists of a secretary, vice-secretary, and one or more additional members. It is the duty of each of these boards of examiners, through the secretaries, to supply the Commission with information in reference to the needs of the service, to give out application blanks and circulars of information to applicants, to receive applications for positions for which no educational examination is

required, to grade the papers, to establish registers of eligibles, and to do such other work as the Commission may direct.

Applications for positions requiring an educational test should be made to the Civil Service Commission at Washington, but applications for positions not requiring an educational test may be made to a local board of examiners, by whom the applicants will be graded according to age, intelligence and experience, character as workmen, and physical qualifications, in accordance with the usual regulations. All marks are given on a ratio of 100, but different weights are attached to the classifications, greater weight usually being given to experience than to other elements. The principal facts to be considered are age, quality of work, rapidity of work, industry, experience, physical condition, and general intelligence or disposition.

Applications for field employment.—The applications for employment in the reclamation service have become so numerous that a printed circular has been issued governing the matter, dated May 7, 1903. This is quoted as follows:

ALL POSITIONS UNDER CLASSIFIED SERVICE.

Positions requiring skilled or technical service, such as engineers, assistant engineers, transitmen, levelmen, plane-table men, and their principal assistants, are under the classified civil service, and appointments to them are made after certification by the Civil Service Commission. Examinations have been held and several large eligible lists established from which selection must be made. Similar examinations will be held from time to time; inquiries regarding examinations should be addressed to the Civil Service Commission, Washington, D. C.

Appeals to prominent men for assistance in securing appointment are useless, as selections from the civil service list must be made without reference to sentiment, charity, politics, or religion, and attempts to influence the action of appointing officers in any such way must necessarily prove unavailing.

FEW ADDITIONAL MEN DESIRED.

When the reclamation law went into effect experienced engineers in the employ of the Geological Survey were detailed to this work. From time to time the force has been increased by adding experienced and qualified men obtained through civil-service examination. The surveying and engineering parties are practically complete, and only a few additional engineers and assistants will be needed, from time to time, to fill vacancies caused by resignations or by gradual expansion of the operations.

NO TEMPORARY EMPLOYMENT.

No college students or other young men are needed for temporary or summer employment. There is a large eligible list of young college men who have been graduated or are about to be graduated, and all young engineers needed as aids or assistants are being selected from this list, prepared by the Civil Service Commission.

UNSKILLED LABORERS.

Teamsters, cooks, and unskilled laborers are employed by the chiefs of field parties, and applications for such positions should be addressed to them and not to officials at Washington. The chief of each field party is held responsible for his men. As a rule each chief of party has a large list of names from which to select.

All the employees last mentioned will, when practicable, be engaged in or near the areas under survey, and they must present themselves at the field outfitting point of the party to which they are attached and will be discharged at the end of the field season. No traveling expenses to and from the field will be allowed. Preference will be given to applicants from States in which the work is to be done.

CORRESPONDENCE.

All official correspondence of the reclamation service should be conducted according to the ordinary rules of business, but it is in addition important to observe certain proprieties or courtesies that tend to facilitate the accomplishment of results. In any large organization there are certain requirements that are due to the necessity of keeping various persons informed of the details of operations.

The chief engineer is the responsible head of the reclamation service, under the Director of the Geological Survey and of the Secretary of the Interior. As far as possible all communications should pass through his office, in order that he may keep himself informed concerning details of the work. All letters from men in the field should pass through the chief of party or district engineer to the chief engineer, in order that the various men responsible for details may be kept fully informed on all points.

All letters from the chief engineer are addressed to the consulting, supervising, or district engineer, or through chiefs to the individuals, if necessary to communicate directly with them. As a rule, direct correspondence with the chief engineer is limited to a relatively few responsible men in the field.

Persons connected with the reclamation service should not correspond directly with other offices or branches of the Government nor with the higher officials of the Department of the Interior. Official rules provide regular channels for the transaction of business, and confusion results if these channels are not followed. No correspondence should be had concerning official plans or duties with public men or members of Congress. All letters received that relate to the general policy or the larger features of the reclamation service will be referred to the Director of the Geological Survey, and through him to the Secretary of the Interior.

Each official letter should relate to one subject only, and that subject should be stated briefly at the top of the letter.

For convenience of filing and reference, all letters and ordinary memoranda should be written upon letter paper of the size furnished by the Department of the Interior, this being 8 inches wide and 10½ inches long.

Letters originating from the Washington office are initialed according to prescribed rules, the initials of the person dictating the letter and of the stenographer being placed in the upper right-hand corner.

Other initials may be placed on the letter indicating that the matter has been considered by other persons.

Every letter covering inclosures should distinctly specify each paper inclosed and note such as are duplicates, and every inclosure should be so marked as clearly to indicate its nature and substance and its relation to the letter of transmittal. The number of inclosures should be plainly stated at the foot of the letter.

Frequently brief letters of inquiry are sent out from the office of the chief engineer to which a few words of answer only are necessary. In such cases the reply should be written (with signature and date) upon the blank space below the letter, which should be returned at once to the chief engineer's office. If the reply can not be given on the lower part of the letter sheet, as many additional sheets should be attached as may be necessary. In this way the original inquiry and reply will be kept together.

Telegrams should be sent on the official blank provided by the Geological Survey for this purpose, and plainly marked "Official business, Government rates." Those addressed to Washington should not be prepaid, and those going from Washington should be received without payment, as all charges are settled at the Washington office.

Telegrams addressed to the chief engineer should be addressed, not by name nor to Chief Engineer, but "Hydrographer, Geological Survey." If addressed to him by name, they may be delivered at his residence and not at the office.

Telegrams between officers in the field, if sent from the larger telegraph offices, can be charged to the Government and the account settled at Washington. To guard and facilitate such charges, both the Western Union and Postal Telegraph companies have issued cards to the principal engineers to the effect that telegrams signed by the persons to whom these cards are issued, and answers thereto, when indorsed "Government business," will be transmitted on the lines of the companies at Government rates on account of the United States. Payment for telegrams handled in accordance with the cards will not be required of the sender.

It is sometimes difficult to convince the operators at smaller stations of the propriety of this course of procedure, and if the telegrams will not be received in this way it is preferable to prepay them at Government rates rather than subject the recipient to the trouble of making payment or running the risk of delay.

REQUISITIONS.

Requisitions for instruments and supplies should be made upon separate sheets from other communications, and should not be included with correspondence relating to other subjects.

Requisitions should be classified according to material; instruments, for example, should be grouped together in one request, and blanks, notebooks, and similar supplies in another, so that the separate requisitions may be forwarded to the separate divisions that will supply the material.

Each person filling a requisition is required to indicate by initials or otherwise the fact that the requisition is filled, and to mark the package with his initials, so that if it is received in bad order the responsibility can be placed. Persons in the field receiving material not in good condition should immediately report the fact to the chief engineer.

Requisitions for instruments should state the nature of the work to be performed and the character and quality of the instruments desired, in order that the person in charge of supplies and equipment may ascertain whether instruments suitable for the purpose can be furnished from the supplies on hand or available.

If requisitions are not promptly filled or acknowledged, the attention of the chief engineer should be called to the matter by a statement that a requisition for certain material was sent on a certain date. Care should be taken to prevent duplication and to indicate that the second request refers to a requisition previously sent.

REPORTS.

Frequent and definite reports concerning the field work must be made to the chief engineer by district engineers and by men in the field, especially by chiefs of parties and men acting independently. Intelligent control and direction of work is possible only through the definite information obtained by means of such reports, and men in the field must show from time to time what has been accomplished for the expenditures made. But it must be remembered that while on the one hand the field work can be better understood and greatly facilitated if concise, definite statements are periodically made, yet it is, on the other hand, desirable to avoid delaying field work by the preparation of reports more elaborate than absolutely necessary.

Information to the public should be given out only through official channels. On this point the following circular has been issued, dated July 1, 1903:

> The attention of engineers and assistants in the reclamation service, and of all other persons concerned, is called to the provisions of section 2 of the law of June 17, 1902 (Stat. L., vol. 32, p. 388), which reads as follows:
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> That the Secretary of the Interior is hereby authorized and directed to make examinations and surveys for, and to locate and construct as herein provided, irrigation works for the storage, diversion, and development of waters, including artesian wells, and to report to Congress at the beginning of each regular session as to the results of such examinations and surveys, giving estimates of cost of all contemplated works, the quantity and location of the lands which can be irrigated therefrom, and all facts relative to the practicability of each irrigation project; also the cost of works in process of construction as well as of those which have been completed.

This section indicates clearly the manner in which information concerning these surveys and examinations is to be given to the public. All persons desiring information on the subject should be referred to the annual report printed by authority of Congress or to the Secretary of the Interior. The necessity for exercising prudence in this matter arises from the fact that the Secretary of the Interior, in carrying out the purpose of the law, must from time to time secure certain lands or privileges essential to the construction of the reclamation works. In order to obtain these lands and rights at reasonable market rates it is necessary to mature plans quietly and to make arrangements with the owners upon a fair and equitable basis. If a report is spread that enormous sums are to be expended in a given locality, prices are advanced to a point where it is impossible to deal with the owners without condemnation proceedings. It is intended that facts shall be obtained for the information of the Secretary only. These will be passed upon by experts, who will consider them from various standpoints and without personal prejudice or sentiment. The results, together with recommendations, will be brought to the attention of the Secretary, and individuals should not discuss the policy nor anticipate the action of the Secretary by announcing plans or stating what may be done in the future.

If the Secretary believes that he can properly give to the public the results of examinations and surveys he will do so upon request.

Reports are required at different periods for different purposes; as weekly, monthly, quarterly, and annually.

Weekly reports.—A weekly report, covering the operations for each day, should be submitted to the district engineer by the chief of each field party, and by each man working alone. This report should be concise, and should state for each day the essential facts as to the character and progress of the work. Such statements as "engaged in computation," or "measuring streams," or "running level lines," are not sufficient; a few additional words must be added, specifying where and for what purpose the particular work is being done, so that the district or supervising officer may obtain at a glance a comprehensive idea of the work in hand without the necessity of referring to letters or correspondence. The report should conclude with a few words as to the probable operations of the next week, and should be mailed as promptly as may be practicable, consistent with the demands of the work.

Monthly reports.—The monthly report should give a résumé of the work accomplished during the month by each party, or in each district, the facts being assembled in concise order for quick reference, so that it may be perfectly plain what work is being carried on and where. This report should show the rate of progress, the present conditions, and the plans for the coming month. The monthly report should be forwarded with vouchers, or a statement of classified expenditures, to the office of the chief engineer for his information. It should be accompanied by the weekly reports of field parties, if these have not been abstracted in the report.

A duplicate or carbon copy of the monthly report should be forwarded to the supervising engineer for his information. The original report sent to the office of the chief engineer may be accompanied by

reports of the various chiefs of parties, construction engineers, etc., and these can be marked for return to the district office if it is not considered desirable to retain a copy. Care should be taken to avoid unnecessary copying and to forward papers for examination and return where such duplication may lead to unnecessary delay or expense.

Equal care must be exercised to forward full information of value for preservation. It has been found that valuable data collected by various hydrographers and engineers have been lost or overlooked or have not been made sufficiently prominent in general correspondence. Each man should be held responsible for transmitting facts which may come under his observation even though these are negative in character, namely, that certain streams were found to be dry or that surveys have not been completed in certain areas.

All vouchers for the pay of occasional employees and men not attached to field parties should be accompanied by the daily statement, on Form 9–245, showing the fact that work has actually been performed on these days and indicating the character of the work. This statement must be brief yet explicit; for example, it should state not simply "computing," but should indicate the character and purpose of the computations.

Quarterly reports.—The quarterly reports are largely administrative in character, the quarter having been selected as the most convenient unit for accounting for property and discussing finances. They should comprise a résumé of expenditures incurred, a statement of the character of these, and a report of the results accomplished, in units of miles of level or transit line, square miles of topography, yards of construction, or other definite, intelligible units.

The quarterly statements and accounts are of first importance from the administrative standpoint. They consist primarily of two parts, the first for transmittal by the chief engineer, through the Director to the Department, and the second the portion for the information of the chief engineer and for the records of his office. The first part of this quarterly statement and estimate consists of descriptive matter in somewhat stereotyped form, giving under each project or principal heading one or more paragraphs devoted to each of the following subheadings:

Location.
Past operations.
Present conditions.
Future work.
Expenditures.

Under the head of "Location" should be given a brief description of the location of the work, such that a stranger taking the material can gain some idea of how the work is being carried on.

"Past operations" should include the work performed, its extent, scale, and the results accomplished.

"Present conditions" should recite the character of the work in hand during the current quarter in which the report is prepared.

"Future work" should cover the operations proposed for the next quarter, or those for which authority to continue the work is desired.

"Expenditures" should recite briefly the total expense incurred up to the end of the preceding quarter; also the probable expenditures for the current quarter and the amount which should be set aside to cover the future work for the ensuing quarter.

This first part should be sent in promptly, so as to reach the chief engineer on or before the 15th day of March, June, September, and December. It should be prepared in the form in which it is to go to the Department, so that it can be assembled quickly and copied without the necessity of editing or extensive revision.

There should also be prepared, if possible before the above descriptive material, an estimate showing the cost of the work done during the quarter, the units of miles surveyed or yards of construction completed, the rations issued, and the cost of other important items. The information thus set forth is to be forwarded to the chief engineer for the records of his office, and the results should be briefly summed up in the material which is to go forward to the Department.

Another important quarterly operation is in connection with the property returns. Every reasonable effort should be made to dispose of material not in use, and to keep the property returns explicit. All property should be inventoried and accounted for, all worn-out and useless articles being condemned or sold after the necessary authority has been obtained. At the end of the calendar year especial attention should be given to cleaning up the property returns, no unnecessary articles being kept; at the same time an account should be rendered for all material obtained or on hand.

Annual reports.—Four printed annual reports are required by law and by custom, and the material for these must be prepared promptly and transmitted to the chief engineer on the days specified below.

1. The Director of the Geological Survey reports to the Secretary of the Interior at the end of the fiscal year all of the operations of the Survey from July 1 to June 30. The portion of his report relating to the reclamation service contains from 5,000 to 10,000 words, and the work done during the fiscal year on each project or survey should be described in from 200 to 500 words. These descriptions should be sent to the chief engineer on or before June 1, so that they may be available for use before June 15.

2. The Secretary of the Interior also makes an annual report at the close of the fiscal year, and in this report reference to the reclamation service is restricted to less than 1,000 words. Reference to work in

each State or Territory should therefore be confined to 100 words or less. The material for this report should be sent to the chief engineer on or before July 15.

3. Under the terms of the reclamation law, the Secretary must also make a specific report to Congress at the beginning of each regular session, on the first Monday in December. This report must be full and complete, as required by law, and must state every detail of cost and results. It must be prepared during the latter part of the field season and must state concisely what has been accomplished during that season. The material for this report—the annual report of the reclamation service—must be derived from full reports made by the district and other engineers, and these reports should be sent to the chief engineer on or before October 15.

The manuscripts of these reports should be accompanied by illustrations sufficient to make clear all important points. A table of contents should accompany each manuscript or portion of report, as well as a list of illustrations, giving the title of each illustration and number of the manuscript page at which it should appear. The illustrations may consist of photographs or drawings. Upon each photograph or drawing should be written a descriptive title and the number of page of the manuscript where it is to be used. All drawings to be used for illustrations should be clearly made with a good quality of India ink on white paper; tracings should be avoided if possible. No lettering should be made in ink; words and figures should be plainly written in pencil at the proper places. Titles should be written on the margin in pencil or ink. These should be explicit and should give the number of page of the manuscript where the drawing is to be placed. Drawings for illustrative purposes should be simple; all detail or elaboration which is not absolutely necessary should be avoided. Elaborate lettering is worse than useless, for many drawings are reproduced by processes that require the removal of the original lettering and the substitution of another form. Tracings can be utilized, but are not so desirable as good, clear drawings on white paper. Tinted papers should be avoided, especially those having a yellowish tinge. In this connection, attention is called to the book of Instructions, page 100.

Each manuscript for the annual report of the reclamation service should be accompanied by a short abstract, to be used by the chief engineer in his introduction. This abstract should be concise and should cover in a few pages the essential features and conclusions of the more detailed report. In this report it is essential to give especial attention to the financial statement—to show the amounts expended or liabilities incurred, the results accomplished, and the amount of funds probably necessary for the work of the ensuing year; also all other facts essential to a clear comprehension of the costs and benefits.

4. In addition to the reclamation report it is customary to publish at the end of each year all matter pertaining to the operations of the hydrographers, hydrologists, and others, in separate volumes that present all of the data accumulated during the calendar year. District engineers are expected to see that all data of this kind are assembled and presented promptly to the chief engineer as soon as possible after January 1 of each year, so as to reach him by January 15.

BOOKKEEPING.

The system of bookkeeping for the reclamation fund has been given considerable thought, and an attempt is made to devise methods of allotment of expenditures in the most rapid and economical manner possible. Each voucher, before transmittal in duplicate for payment, must be certified by a responsible officer having full knowledge of the expenditure. He should require his assistants to previously check the items and initial the papers, thus attesting their accuracy.

The voucher, if payable at the Washington office, should be forwarded to the chief engineer, with a charge sheet or letter of transmittal (form 9–272). If payable by a local disbursing officer, the charge sheet should be sent at once to the office of the chief engineer at Washington. These sheets form the basis for bookkeeping and should be filled out to give the essential facts on which the allotment of the various items may be based, thus serving as pages of a day book.

The original voucher goes to the Treasury and the duplicate is retained by the disbursing clerk; the charge sheet alone remains in the office of the chief engineer, and, although the original or duplicate voucher may be consulted, this is attended with some difficulty and loss of time. The charge sheet should therefore contain all the information essential for proper bookkeeping entries.

The object of bookkeeping is to enable a statement to be made at any time of exactly how much money has been expended for various objects and in different kinds of work. There are at least three distinct degrees to which the bookkeeping may be carried, and these, for convenience, may be designated as primary, secondary, and tertiary.

The primary classification of accounts covers the larger allotments, such as to States, to distinct projects, or to reconnaissance surveys. It is of first importance to know the total amount expended under these primary headings.

The secondary distribution of accounts is for the purpose of ascertaining how much has been expended in each State or in each locality for different classes of work. The principal items are the cost of preliminary surveys, topographic surveys, testing for foundations, location surveys, subdivision of land, designing, construction, etc.

The tertiary classification again subdivides these features into the various ways of expenditure. These consist of services, traveling expenses, subsistence, equipment, etc. A wide latitude of judgment may be exercised in this tertiary classification, and the engineer approving the accounts must be guided by general principles rather than by specific instructions.

The following lists give the principal headings provisionally determined upon for the primary, secondary, and tertiary classifications. Suitable blanks are being provided for convenience in stating these items, and it is believed that, with reasonable care, the engineer, in certifying accounts, can apportion the items without notable delay or difficulty.

Primary classification.—The following are the principal headings in the ledger of the Washington office. The charge sheet for each voucher should have one of these headings prominently given, and as far as practicable, no voucher should be assigned to more than one of these principal items:

Washington office:
- Supervision.
- Correspondence.
- Bookkeeping.
- Disbursing.
- Legal expenses.
- Computing.
- Drafting and photography.
- Lithographing.
- Stationery.
- Instruments and supplies.
- Express and telegraph.
- Furniture.
- Messenger.

Arizona:
- Salt River.
- San Carlos.
- Yuma. (See California.)
- Reconnaissance, etc.

California:
- Sacramento Valley.
- Owens Valley.
- Yuma.
- Reconnaissance, etc.
- General.
- Cement pipe.

Colorado:
- Uncompahgre.
- White River.
- Grand.
- Reconnaissance, etc.
- General.

Idaho:
- Minidoka.
- Dubois.
- Payette-Boise.
- Reconnaissance, etc.

Kansas:
- Reconnaissance, etc.

Montana:
- Milk River.
- St. Mary.
- Marias.
- Sun River.
- Reconnaissance, etc.

Nebraska:
- Pathfinder.
- Goshen Hole.
- Gering.
- Reconnaissance, etc.

Nevada:
- Truckee-Carson.
- Reconnaissance, etc.

New Mexico:
- Hondo.
- Urton.
- Rio Grande.
- Reconnaissance, etc.

North Dakota:
- Fort Buford.
- Reconnaissance, etc.

Oregon:
- Umatilla.
- Malheur.

Oregon—Continued.
  Harney.
  Reconnaissance.
  General.
Oklahoma:
  Reconnaissance, etc.
South Dakota:
  Belle Fourche.
  Reconnaissance, etc.
Utah:
  Utah Lake.
  Bear Lake.
  Reconnaissance, etc.
Washington:
  Big Bend.
  Okanogan.
  Reconnaissance, etc.
Wyoming:
  Cody.
  De Smet.
  North Platte.
  Reconnaissance, etc.

Secondary classification.— In the lower part of the charge sheet are to be inserted the amounts of the secondary allotment or distribution of vouchers, this consisting of the items given below. A separate page of the ledger should be assigned to each secondary item, the tertiary classification being extended in columns across the page.

Preliminary survey.
Topographic survey.
Testing or boring.
Location survey.
Subdivision of land.
Designing.
Construction.
Executive.

Tertiary classification.—The tertiary subdivision or character of expenditures is shown under each of the secondary headings. These subdivisions are recognized as being approximate in character, and it is not desirable to make the subdivision too minutely.

The principal items of the tertiary division are services, traveling expenses, subsistence, equipment, office expenses, supplies, storage, etc. It is desirable not to have any items of miscellaneous expenditure if these can be avoided, as it is assumed that all items can be assigned to one or another of these headings.

DISBURSEMENTS.

The next important matter to be considered is the relation of the men to the Government in the matter of accounts. Good financiering is the foundation of success, and although no profit is here to be considered, yet it is important that all work done should be worth what it costs, and that the interests of the Government should be protected in every way. On the other hand, the men who are doing the work must be promptly and fully provided with funds, as all delays or difficulties in this matter are sources of expense and trouble both to the workmen and to the work.

At this time it is not desirable to discuss the question of costs and benefits of the surveys and the subsequent construction, but it is important to consider now some of the features of rendering accounts and of so arranging these that they will be quickly audited. The situ-

ation as regards the reclamation service is in one sense peculiar. All of the work is being performed in remote localities and under difficulties, and is now carried on by men who have had very little previous experience with Government methods. During the past season there have been unusual and vexatious delays and difficulties in securing settlement of accounts; each man in the field has a number of grievances and in his opinion injustice has been frequently done to him.

In order to put the matter of accounts somewhat more clearly before the new men it is desirable to state a few elementary facts or principles which should be known by everyone, but which as a matter of fact are clearly understood by few. Theoretically every man is supposed to know the law, but practically few men do know the law concerning most of the details of Government transactions.

An attempt has been made in the regulations of the Survey—a manual that has now passed through three editions, the last of which was approved to take effect January 1, 1903—and in the Survey's Instructions (approved to take effect May 1, 1903) to outline some of the more important facts and requirements concerning Government accounts. It is impossible, however, in any book of moderate size to discuss or consider all of the details that may come up in current practice, for, as above stated, each man has different experiences, and the difficulties that one man has encountered may not be met by another.

Especial attention should be called to the statements beginning on page 26 of the regulations, in which an attempt is made to present a few instructions covering the more important points. The great difficulty with regard to Government accounts lies in the fact that on the one hand general laws are enacted by Congress, and on the other, rulings are made by the officials of the Treasury Department upon various legal or other points that may arise. In the attempt to apply these general laws and Department rulings to specific cases it is frequently found that they are in apparent conflict or are not applicable to the conditions. Since the general laws do not fit the exact conditions, it is often necessary to state an account in such a way that while it will not run counter to some general law, it will at the same time present the salient facts necessary to obtain its settlement. The auditing officers of the Treasury have little to do with the equity or justice of a claim or an account; they simply pass upon its legality in the light of the facts presented. They may sometimes ignore a legal technicality in the interest of justice or in conformity with common practice, but they can not be expected to do so often.

The general authorization to carry on work covers with it the purchase of ordinary equipment and necessities of field parties, such as horses, wagons, tents, cooking utensils, and articles covered by the ration list of the Geological Survey; also a small amount of customary remedies or medicines needed by men and animals working

at a distance from medical attendance. In this connection care must be taken to explain fully the need of any unusual article of this kind, such as medical supplies, and to keep these to the minimum of actual requirements, since medical attendance is not furnished employees where it can be had in the customary manner.

The theory upon which the present system of accounts of the Geological Survey is based is one that can be understood only after a little explanation. The system has gradually grown up through nearly a generation of practice, and under it millions of dollars have been disbursed.

Under a strict construction of law it is held that no money can in any way be advanced on behalf of the Government. Each purchase or expenditure should be made by a bonded disbursing officer or agent. For operations outside of the city of Washington nearly every Government bureau, except the Geological Survey, has such a bonded disbursing officer with each party or body of men under its direction. In this survey, however, the number of disbursing officers has been gradually reduced, until now practically all disbursements are made from Washington, field men sending all accounts to one or two bonded clerks at that city.

This system, as above stated, is apparently the outgrowth of custom, having received the approval and tacit acquiescence of Department officials throughout many years.

Theoretically the disbursing officer pays for everything, and no Government employee need have money, as his expenses are provided for by the disbursing clerk. Practically, however, the reverse is the case under the present system. The field men must have on hand an ample supply of private funds in order to carry on the Government's work, which they do at their own expense, seeking reimbursement immediately after the end of each month.

The principal exception to this rule is where men in the field purchase supplies, animals, wagons, etc., and arrange with the seller to send signed vouchers to the disbursing clerk at Washington. This can be done with business men who are accustomed to such matters, but frequently the seller is suspicious and will not sign vouchers in advance of payment, or does so only after obtaining the guaranty of the field man that he will be personally responsible for the payment of the accounts.

It thus happens, as above noted, that the men in responsible charge of field parties must have considerable sums of money to meet current expenses, and each man may thus have tied up in Government work from $300 to $500 a month, or even more; in some cases several thousand.

Some of the men are not able to do this without considerable sacrifice, and during the past year a few of them have paid as high as 1 per cent a month to procure funds to be used for Government purposes. Inquiry has been made as to whether these men can not be reimbursed for this outlay, but from the interpretation of the law, as above stated, it is obvious that this is impossible. Relief from this large outlay by employees can be had only by following more closely the provisions of law by appointing bonded disbursing agents for each of the large parties. Gradual changes in this direction are being made, and it is planned to have a disbursing agent in each district to perform ordinary clerical work in connection with his disbursements, and to make himself generally useful to the district engineers. It is desirable, wherever possible, that this clerk should be also a stenographer in order that he may be of greater assistance.

The fact that men are compelled to advance their own money for Government work makes it very important to secure prompt settlement of their accounts. The delays of the past summer have been due to two causes: First, most of the men making the accounts were inexperienced in this work; and second, the force in the Washington office employed to examine and revise the accounts and present them for payment has been inadequate. Both of these causes of delay have been obviated in part; the men have gained larger experience and the office force has been increased.

At the present stage of the reclamation service, as above noted, all matters are in a condition of transition or evolution. The necessity of having competent disbursing officers at each important locality where work is being carried on is now apparent, both for economy and for full compliance with the spirit of the law. In view of such changes, attention is called to certain general requirements:

The chief engineer supervises all disbursements by officers of the reclamation service. His approval of their accounts, as to the necessity or expediency of the expenditure and the prices paid, and his certificate that the mode of expenditure adopted was the most economical and advantageous to the Government are requisite. If the district or disbursing officer certifies that the mode of expenditure adopted was the most economical and advantageous to the Government, the certificate of the chief engineer to that effect may be omitted.

As has been already stated, the chief engineer presents to the Secretary of the Interior, through the Director of the Geological Survey, on or before the last day of each quarter a report of the operations of the reclamation service, with such suggestions relative to its affairs as he may deem expedient, accompanied by an estimate of funds required for the service during the ensuing three months.

An officer or agent who may supervise the disbursement of money for any work of construction or survey in his charge, or any operations

involving expenditures, must furnish to the chief engineer an estimate of the probable expenditures for the coming quarter and a plan showing the manner in which it is proposed to apply the money.

All officers, agents, or other persons who are charged with the safe-keeping, transfer, or disbursement of the public moneys, shall keep an accurate entry of each sum received and of each payment or transfer, and shall render distinct accounts of the application thereof according to the appropriation under which the moneys may have been advanced to them. Every officer or agent who, having received public money which he is not authorized to retain as salary, pay, or emolument, fails to render his accounts for the same shall be deemed guilty of embezzlement, and shall be fined in a sum equal to the amount of the money embezzled, and shall be imprisoned not less than six months nor more than ten years.

Every officer or agent of the United States who receives public money which he is not authorized to retain as salary, pay, or emolument, shall render his accounts monthly * * * (Rev. Stat., 3622.)

The law requires that a separate account be kept with each appropriation disbursed. The usual forms of account current and abstracts prepared for this purpose and approved by the Comptroller of the Treasury should be used by disbursing officers. Accounts current should be made in triplicate. Two copies, accompanied by two sets of abstracts and one set of vouchers complete, should be forwarded to the chief engineer. The other copy, also accompanied by abstracts and vouchers, should be retained by the officer.

An error made in an account must be corrected in the next account current of the officer after he is informed of the error, and reference should be made therein to the particular voucher in which the error occurred.

In notifying officers of remittances the chief engineer will inform them of the amount remitted under each head of appropriation. All officers who make transfers of public funds will pursue a similar course, stating in the invoices and receipts the amount transferred from each appropriation. In the letters notifying officers of requests for remittances the chief engineer, in addition to the amount of each appropriation, will give the titles of appropriations to be used in accounts current, abstracts, and vouchers.

Disbursing officers will promptly, at the close of business at the end of each month, make and forward statements showing explicitly where their funds are deposited. Officers will include in the sums claimed to be on deposit only such as have been officially credited to them, and of which credit they have been duly informed by the depositary.

When an officer is relieved from duty on any work he should certify the outstanding debts to his successor, and transmit an account of them to the chief engineer. Unless otherwise ordered, he will turn over to his successor the public money and property and the books and papers appertaining to the service from which he is relieved.

Disbursing officers, when they have the money, shall pay cash, and not open an account.

When disbursing officers draw checks in payment of accounts on funds placed to their credit with assistant treasurers or other depositaries of the United States, they should note upon the receipt taken for such payment the number, date, and amount of the check given in payment, and designate the assistant treasurer or depositary upon whom it is drawn; and when an account is paid in part by currency, the amount of the same should be stated.

Persons employed in the reclamation service may be allowed actual traveling expenses for travel on duty under orders, to be paid from the allotment applicable to the works upon which such persons are employed. The affidavit of the person should be made in the prescribed form.

Vouchers for purchases made should specify the quantity and price of each article bought, the name and business place of the person from whom it is procured, and the date and manner of purchase. Vouchers for services rendered should state the nature and period of service, the rate of pay per diem or month, etc. Vouchers for emergency purchases or services should state that the price paid was the lowest market rate, and that the mode of expenditure adopted was the most economical and advantageous to the Government.

When original vouchers can not be furnished with accounts, copies duly certified as true by a disinterested officer may be accepted. If no other officer than the payor or payee is at the locality when payment is made, both must certify to that fact and also to the correctness of the copy.

When vouchers are not sent with the account to which they belong, an explanation must be made as to why they were not produced with and included in the proper account.

At the time vouchers are transmitted by an officer with his accounts if any of the articles purchased as noted thereon have been "expended and applied to the purpose for which purchased," a certificate to that effect may be made, and when so made the articles referred to need not be taken up on abstracts of property or on property returns.

Requests for the admission free of duties and charges of articles imported for the use of the Government of the United States, must be accompanied with a list of the articles to be forwarded to the collector of customs for his information and guidance and for the permanent files of his office.

DISALLOWANCES AND SUSPENSIONS.

One of the most important duties of an officer of the Government is to learn how to make acceptable accounts. Much experience is

required to do this, and much application of constructive energy. It is our business to learn to do this clerical work rapidly and economically. The Treasury officials at Washington are, in one sense, not constructive as regards such matters. Their rulings are mainly prohibitions—this or that thing should not be done. On the other hand, there are few persons whose business it is to tell the field man how to do things. It becomes, therefore, one of the important duties of the more experienced field men to give to others the benefit of their experience and to show them how Government accounts can be rendered acceptably, accurately, so that they may stand every test of scrutiny or of searching investigation.

Occasionally mistakes are made by new men, and disallowances of just claims result; but with care and forethought it is usually possible to avoid such difficulties. During the past season each man has been perplexed by his accounts and has suffered financially, but as a rule the losses have been comparatively trivial by reason of the fact that every effort is made at the Washington office to assist in rectifying blunders and supplying omissions. The accounts that the men make in the field go at first to the office of the chief engineer in Washington, and are there carefully scrutinized for mistakes and informalities by clerks who have spent years in this particular work and who are guided by the monthly statement of suspensions and disallowances made by the Auditor's office.

A number of questions have been asked concerning disallowances on accounts. As a rule an attempt is made to notify men in the field of the reason for a disallowance, but in some cases the reason is so obvious that it should not be necessary to write a letter.

As far as practicable all mistakes and clerical errors are detected during the examination of accounts in the office of the chief engineer. These are briefly pointed out or explained to the men in the field. From this office the accounts go to the disbursing clerks of the Geological Survey, who also examine them and if they are satisfactory make payment by checks. It is assumed that if errors are discovered later, in the Treasury Department, and disallowances made for any items, the men in the field will reimburse the disbursing clerk for the amounts covering items disallowed. Thus it frequently happens that after an account is paid the disbursing clerk will call upon the field man to refund a small amount which has been suspended or disallowed by the accounting officers of the Treasury Department because of an error discovered by them. Sometimes satisfactory explanation can be made by the man in the field, and it will not then be necessary to refund the money, but if he can not give such explanation he should immediately forward the money to the disbursing clerk; otherwise, it will be necessary for the certifying officer to pay this amount out of his own pocket.

Disallowances are of two kinds; those made by the disbursing clerk before the account is paid and those made by officers of the Treasury Department after the account has been paid by the disbursing clerk. In the first case the deduction is made by the disbursing clerk and a check is sent for the remainder of the account. In the second case the money must be refunded to the Treasury Department by the field man through the disbursing clerk. A careful record is kept of these matters and a definite system is followed.

The accounts of the disbursing clerks or agents are sent to the Treasury Department and are there examined carefully for errors. These examinations may not be made for several weeks or months. Hence, an account for January, which may be paid in February, will probably not be heard from until May or June. At that time the disbursing clerk will receive from the Treasury Department a letter allowing most of the items in his account, but suspending or disallowing certain other items. An example of these suspensions is given in the following paragraphs, extracted from one of the Auditor's letters relating to accounts for December, 1902. In these the names of the persons concerned have been omitted.

SAMPLES OF SUSPENSIONS OR DISALLOWANCES MADE BY THE AUDITOR IN ACCOUNTS OF A DISBURSING CLERK.

Vou. 40, subvoucher 12, includes charge for supper, November 3, 1902, 50 cents. Suspended, as supper for same date is charged per voucher 20 ..... $0.50

Vou. 41, subvoucher 1, W. U. Telegraph Co., message of 57 words from Rawlins, Wyo., to Los Angeles, Cal., 75 cents. Should be 71 cents. Suspended. .04

Vou. 49, subvoucher 7, for $3.00; receipt is for $1.50 only. Suspended 1.50

Vou. 85, notary fee, 50 cents. Legal rate for Idaho, 25 cents. Suspended ... .25

Vou. 103, subvoucher 15 of Wells, Fargo & Co., express, $5.16. Receipted for $5.15. Suspended ..... .01

Vou. 104, subvoucher 2, board bill at Denver, Colo., $2.50. Suspended, as travel order does not authorize stop-over at Denver 2.50

Vou. 119, subvoucher 10, ——— ———, services as cook from November 1 to 6, 1902, $9. Suspended, as he was paid for month of November, per voucher 189 of this account 9.00

Vou. 232, subvoucher 15, lunch ——— ——— ——— ———, Dec. 10, charged 10 cts. Suspended, as dinner and supper for same date are also charged per subvouchers 16 and 17 ..... .10

Vou. 56, ——— ———, first item, 50 lbs. flour, at 1¼ cts. per lb., $1.25; should be 63 cts. Suspended ..... .62

Vou. 68, December 2, 1902, sleeper, Reno to San Francisco, $2. Correct charge, $1.50. Suspended ..... .50

Vou. 69, ——— ———, Nov. 25, 1902, railroad fare, Roseville to Sacramento and return, $1.10. Disallowed, travel being over bond-aided line 1.10

Vou. 156, ——— ———, p. 1, line 3, two fry pans, at 20 cts., 80 cts.; should be 40 cts. Suspended ..... .40

Vou. 178, ——— ———, subvoucher 6, breakfast, Dec. 8, charged 19 cents. Suspended, as breakfast for same date is also charged for subvoucher 24, January voucher 134 ..... .19

Vou. 68, January 6, 1903, porter's fee, 75 cts.; should be 25 cts. Suspended.. .50

In some of the cases above given the apparent error can be explained and the suspension will be removed; in other cases, especially those involving a slight difference in car fare, it is preferable to remit the few cents, and let the amount be disallowed. It must be remembered that some one refunds this money to the Treasury—either the man in the field, who, presumably, made the mistake, or the certifying officer.

AUDITOR'S CIRCULAR.

The following paragraphs regarding disallowances and suspensions are taken from a Treasury Department circular dated April 18, 1902, and signed by R. S. Person, Auditor for the Interior Department:

Disbursing officers, disbursing clerks, and disbursing agents of the Department of the Interior, and special agents employed by the Secretary of the Interior charged with the disbursement of public moneys are instructed to observe strictly the following regulations in making explanations to statements of differences made in the examination and settlement of their accounts in the Treasury Department:

DISALLOWANCES.

Explanations to items *disallowed* by the Auditor on examination and settlement of an account can be considered only by the Comptroller of the Treasury on revision of the account. Requests for revision of accounts should be made directly to the Comptroller of the Treasury, and in all cases within one year from the date of settlement.

SUSPENSIONS.

1. All explanations of items *suspended* by the Auditor on examination and settlement of an account must be made directly to the *Auditor for the Interior Department.* (Treasury Department circular 87, issued by the Comptroller of the Treasury and approved by the Secretary of the Treasury, April 25, 1895.)

2. Explanations should be made promptly. They should reach the Auditor, if possible, not later than *thirty days* from date of settlement.

3. Different accounts must be explained in separate communications, so that the explanations may be filed with the certificate of settlement in which final action is had.

4. Refer to accounts by period covered, and Auditor's certificate number. It will facilitate the handling of your explanation if you will begin it in the following manner:

Herewith are submitted explanations to items suspended in my account as ——— for the quarter ended ———, 190—, stated per Auditor's certificate No. ——.

Follow this with the explanation to each item separately and in regular order, referring to voucher number, and giving amount claimed under each item.

5. Explanations should answer fully and specifically all questions asked by the Auditor, and objections raised to the allowance of any item. All the items suspended in an account must be explained at the same time, so far as possible.

6. Items to which you desire to make no explanation should be indicated in your communication as "conceded."

7. Where explanations are not received within a period of six months from date of settlement, and no satisfactory reason appears for the delay, it will be assumed that the disbursing officer has no sufficient explanation to make, and the item will be disallowed.

The attention of field men is called to the fact that the Government spares no expense to correct trivial errors. All bills are rigidly

checked, and if errors are found much correspondence is required to correct them. The Government may incur an expense of ten dollars, or a hundred dollars, in clerical services, correspondence, etc., to correct an error of a few cents in a voucher. For this reason it is extremely important that anyone initiating an account should see that it is correct in the first instance. Errors may not be detected for months or even years, but when detected the men in the field are called upon for explanation. Thus it is to the interest of all concerned to use every possible precaution to avoid the necessity of taking up these trivial matters a second time.

EXPRESS CHARGES.

Engineers and hydrographers are requested to give especial attention to matters pertaining to shipment of material by express. The experience of the past summer has shown that it is costing the Government more to get satisfactory vouchers than the expressage itself amounts to. This is due to the fact that the regulations are very exacting and require certain information which it is extremely difficult to obtain without large outlay of time and money in clerical labor and in visiting the offices of express companies.

The first rule to be observed is that material should not be sent by express if it can be sent by mail or freight. In this connection the rule regarding the use of the official frank, contained on page 19 of the Regulations, should be noted. Material too bulky to be sent by mail should be sent by freight if time permits. On this point the Regulations, page 51, section 68, and Instructions, page 34, section 12, should be consulted.

Annoyance arising from defective vouchers can be avoided by utilizing the survey bill of lading (Form 9–060), both for express and for freight. The principal express and freight companies have instructed their agents to accept this form, but trouble will usually be found with agents at out-of-the-way stations. Some of these men refuse to accept this bill of lading, and occasional instances have arisen where agents have insisted upon cash payments in addition to the bill. When the survey bill of lading is used payment should not be made. If the agent refuses to accept the bill of lading, the facts should be briefly reported to this office by telegram and instructions should be asked if the place is near a telegraph office. If it is not near a telegraph or post-office, payment should be made under protest, and the facts should be stated upon the voucher.

When express matter is received great care must be taken to have the voucher properly filled out. This is a comparatively small matter if it is possible to visit the express office and present the vouchers in person, but where, as is frequently the case, the vouchers must be sent

and obtained through the service of a messenger, a teamster, or an uneducated person, care must be exercised in writing out full instructions as to how the vouchers should be filled. The rules require for the ordinary field account two distinct receipts, one of these being in duplicate, making three signatures in all. These signatures must be in ink, not in pencil, nor in the so-called indelible pencil commonly used by express messengers. The signature must be clear and legible, and not merely a hasty scrawl, as is frequently the case.

The first voucher required is the ordinary slip receipt supplied by the express company. This must contain all of the facts furnished by the company. In addition to this ordinary slip receipt it is necessary to obtain duplicate subvouchers on the usual form. These subvouchers must state distinctly the facts called for in the regulations and instructions, and also certain additional facts recently required, particularly the rate per hundred. Express shipments are not computed directly on the weight of the article, but upon a basis of so much per hundred pounds between certain points. For example, if payment on a 7-pound package is 35 cents, the rate is not 5 cents per pound, but some undeterminable rate per hundred, which can be ascertained only from the express company.

The principal facts required to be stated upon an express subvoucher are as follows:

Point from which material is shipped.

Point to which material is shipped.

Name of express company first receiving goods.

Name of express company delivering goods.

General statement as to contents of package.

Statement as to value. (This should not exceed $50, even though value is several hundred dollars.)

Condition in which received—whether good or bad.

Rate per hundredweight.

It has recently been decided that when express is paid in connection with traveling expenses it is not necessary to secure subvouchers on the Geological Survey form. A single receipt from the express company giving the above facts, on its ordinary form, will suffice. This will greatly simplify the labor and expense of procuring the necessary vouchers, and it is hoped that this rule will prove operative in the future.

In order to avoid making mistakes, it is essential that the men in the field should know something about the methods of express companies. There are a number of these companies—Adams, American, Denver and Rio Grande, Northern Pacific, Pacific, Southern, United States, Wells-Fargo, etc. Most of these have adopted a uniform method of classification and uniform tables of graduated charges per hundredweight (see pp. 64, 65).

In brief, it may be said that when the rate per hundredweight is less than $2, the charge increases gradually by units of 5 pounds, but not in exact proportion to the weight. For example, a 1-pound package will be 25 cents, whatever the rate; 2 pounds may be 25 or 30 cents; 3 pounds from 25 to 40 cents; 5 pounds will run from 30 to 55 cents, and 50 pounds from 40 cents to $1.10.

When the rate is above $2 per hundredweight a pound rate is made for 50 pounds and over. For packages under 50 pounds in weight the graduated charge is sometimes greater than the proportional weight would indicate.

Two packages sent by the same person to the same destination are not charged on a basis of their united weight, but are each charged separately, if under 20 pounds each. It is important therefore, for economy, to make one package of material as far as practicable.

Graduated charges, as above described, apply to matter weighing less than 100 pounds when the rate is under $2 per hundred pounds; also to matter less than 50 pounds when the rate is $2 or more per hundred pounds. For example, between points where the rate per hundred pounds is $1.50, a box weighing 63 pounds must be charged at the graduated rate under the heading "$1.50," or at $1.25. Packages exceeding 7 pounds, carried by more than one company, have a graduated charge for each company.

When the rate per hundred pounds is $2 or more the pound rate is charged for 50 pounds or over. The charge for less than 50 pounds is made by the graduated scale and must not exceed the charge for 50 pounds.

When the through charge can not be accurately ascertained, the charge to the transfer point only should be obtained and the consignee notified of this fact.

In making out vouchers, if the rate per hundred pounds can not be ascertained from the express company, it should be computed from the accompanying table, and the error, if any, should appear to be in favor of the Government, for the reason that errors which appear to favor the Government are not corrected by the accounting officers, while errors resulting in an apparent overcharge to the Government, however trivial, are sought out, and if discovered refund is demanded.

The express business is comparatively simple when it is handled by a single company, but difficulties arise when express matter is sent over two or more lines. The attempt is made by the company receiving the matter to send it as far as possible over its own line, and it may thus be sent by a route which is not the quickest or the cheapest. Care should therefore be taken to see that, whenever practicable, express matter is sent by a route involving the least possible charge, as the officials of the Treasury Department will suspend express bills on packages that are not sent by the cheapest route.

Graduated charges for packages weighing less than 100 pounds.

[When the rate between any two points is not given below, use the next higher rate for making price.]

	When rate is—												
	.40	.50	.60	.75	1.00	1.25	1.50	1.75	a 2.00	2.50	3.00	3.50	4.00	4.50
Packages not over.....	1 lb. 25	1 lb. 25	1 lb. 25	1 lb. 25	1 lb. 25	1 lb. 25	1 lb. 25	1 lb. 25	1 lb. 25	1 lb. 25	1 lb. 25	1 lb. 25	1 lb. 25	1 lb. 80
Over 1 lb. "	2 " 25	2 " 25	2 " 25	2 " 30	2 " 30	2 " 30	2 " 30	2 " 30	2 " 35	2 " 35	2 " 35	2 " 35	2 " 35	2 " 35
" 2 " "	3 " 25	3 " 25	3 " 25	3 " 30	3 " 30	3 " 35	3 " 35	3 " 40	3 " 45	3 " 45	3 " 45	3 " 45	3 " 45	3 " 45
" 3 " "	4 " 25	4 " 25	4 " 30	4 " 20	4 " 35	4 " 35	4 " 40	4 " 45	4 " 50	4 " 55	4 " 60	4 " 60	4 " 60	4 " 60
" 4 " "	5 " 25	5 " 25	5 " 30	5 " 35	5 " 40	5 " 40	5 " 45	5 " 50	5 " 55	5 " 60	5 " 65	5 " 70	5 " 70	5 " 75
" 5 " "	7 " 30	7 " 30	7 " 35	7 " 35	7 " 40	7 " 45	7 " 50	7 " 55	7 " 60	7 " 70	7 " 75	7 " 80	7 " 85	7 " 90
" 7 " "	10 30	10 30	10 35	10 40	10 45	10 50	10 55	10 60	10 70	10 75	10 80	10 90	10 100	10 100
" 10 " "	15 30	15 30	15 35	15 40	15 45	15 55	15 60	15 65	15 75	15 85	15 90	15 100	15 110	15 115
" 15 " "	20 30	20 30	20 35	20 40	20 50	20 60	20 70	20 75	20 85	20 100	20 110	20 120	20 125	20 130
" 20 " "	25 35	25 35	25 40	25 45	25 55	25 65	25 75	25 85	25 100	25 110	25 120	25 130	25 140	25 150
" 25 " "	30 35	30 40	30 45	30 50	30 60	30 70	30 80	30 90	30 100	30 115	30 130	30 150	30 160	30 170
" 30 " "	35 40	35 40	35 45	35 50	35 65	35 75	35 85	35 100	35 100	35 125	35 140	35 160	35 170	35 190
" 35 " "	40 40	40 40	40 50	40 55	40 70	40 80	40 90	40 100	40 100	40 125	40 150	40 175	40 185	40 200
" 40 " "	45 40	45 40	45 50	45 60	45 75	45 90	45 100	45 100	45 100	45 125	45 150	45 175	45 200	45 225
" 45 " "	50 40	50 45	50 55	50 60	50 80	50 100	50 100	50 100	50 100	50 125	50 150	50 175	50 200	50 225
" 50 " "	55 40	55 50	55 60	55 65	55 85	55 100	55 110	55 110	.....	.....	.....	.....	.....	.....
" 55 " "	60 40	60 50	60 60	60 70	60 90	60 110	60 120	60 120	.....	.....	.....	.....	.....	.....
" 60 " "	65 40	65 50	65 60	65 75	65 90	65 115	65 130	65 130	.....	.....	.....	.....	.....	.....
" 65 " "	70 40	70 50	70 60	70 75	70 100	70 125	70 140	70 140	.....	.....	.....	.....	.....	.....
" 70 " "	75 40	75 50	75 60	75 75	75 100	75 125	75 150	75 150	.....	.....	.....	.....	.....	.....
" 75 " "	80 40	80 50	80 60	80 75	80 100	80 125	80 150	80 160	.....	.....	.....	.....	.....	.....
" 80 " "	85 40	85 50	85 60	85 75	85 100	85 125	85 150	85 170	.....	.....	.....	.....	.....	.....
" 85 " "	100 40	100 50	100 60	100 75	100 100	100 125	100 150	100 175	.....	.....	.....	.....	.....	.....

a When the rate per 100 pounds is $2 or more, charge pound rates for 50 pounds or over, but the charge on a package less than 50 pounds in weight must not be greater than the charge for 50 pounds. If less than $2 per 100 pounds, a graduated charge must be made, but the charge for a shipment at a rate of less than $2 per 100 pounds shall not be more than the charge on the same shipment when the rate is $2 or more per 100 pounds.

When the rate exceeds $20 per 100 pounds, charge pound rates, but never less than the $20 graduated charge for same weight.

Graduated charges for packages weighing less than 100 pounds—Continued.

	When rate is—													
	5.00	6.00	7.00	8.00	9.00	10.00	11.00	12.00	13.00	14.00	15.00	16.00	17.00	18.00	20.00
Packages not over...	1 lb. 30	1 lb. 30	1 lb. 30	1 lb. 30	1 lb. 30	1 lb. 30	1 lb. 30	1 lb. 30	1 lb. 30	1 lb. 30	1 lb. 35	1 lb. 35	1 lb. 35	1 lb. 35	1 lb. 40
Over 1 lb. " ...	2 " 35	2 " 35	2 " 35	2 " 35	2 " 35	2 " 35	2 " 35	2 " 35	2 " 35	2 " 35	2 " 40	2 " 40	2 " 45	2 " 45	2 " 50
" 2 " " ...	3 " 45	3 " 45	3 " 45	3 " 45	3 " 45	3 " 45	3 " 45	3 " 45	3 " 45	3 " 45	3 " 50	3 " 50	3 " 55	3 " 60	3 " 60
" 3 " " ...	4 " 60	4 " 60	4 " 60	4 " 60	4 " 60	4 " 60	4 " 60	4 " 60	4 " 60	4 " 60	4 " 65	4 " 65	4 " 75	4 " 75	4 " 80
" 4 " " ...	5 " 75	5 " 75	5 " 80	5 " 80	5 " 80	5 " 80	5 " 80	5 " 80	5 " 80	5 " 80	5 " 85	5 " 85	5 " 90	5 " 95	5 " 100
" 5 " " ...	7 100	7 100	7 100	7 100	7 100	7 100	7 100	7 100	7 100	7 110	7 115	7 115	7 125	7 140	7 150
" 7 " " ...	10 110	10 115	10 120	10 125	10 135	10 150	10 150	10 150	10 150	10 150	10 165	10 165	10 175	10 180	10 200
" 10 " " ...	15 125	15 135	15 150	15 160	15 175	15 200	15 200	15 215	15 215	15 215	15 235	15 250	15 260	15 275	15 300
" 15 " " ...	20 140	20 165	20 175	20 200	20 200	20 250	20 250	20 275	20 275	20 285	20 300	20 320	20 340	20 360	20 400
" 20 " " ...	25 160	25 185	25 200	25 225	25 250	25 300	25 325	25 350	25 350	25 350	25 375	25 400	25 425	25 450	25 500
" 25 " " ...	30 175	30 210	30 250	30 275	30 300	30 325	30 375	30 400	30 400	30 420	30 450	30 480	30 510	30 540	30 600
" 30 " " ...	35 200	35 250	35 275	35 325	35 350	35 375	35 425	35 475	35 475	35 490	35 525	35 560	35 595	35 630	35 700
" 35 " " ...	40 225	40 275	40 325	40 350	40 400	40 425	40 475	40 525	40 525	40 560	40 600	40 640	40 680	40 720	40 800
" 40 " " ...	45 250	45 300	45 350	45 400	45 450	45 475	45 525	45 575	45 585	45 630	45 675	45 720	45 765	45 810	45 900
" 45 " " ...	50 250	50 300	50 350	50 400	50 450	50 500	50 550	50 600	50 650	50 700	50 750	50 800	50 850	50 900	50 1000

All express matter sent from Washington is prepaid, and all express matter sent to Washington should be sent "collect," so that difficulties concerning shipments to and from this point are avoided. The chief annoyances relate to express packages sent from one party in the field to another. When express matter is shipped a receipt should be taken and forwarded at once by mail to the person to whom the package is directed. The principal facts as to the route and charges should be learned, and the consignee should be notified of these so that he will have no difficulty in making out his accounts.

An extra charge is made for the shipment of merchandise that is valued on a bill of lading at more than $50. As a rule, most of the packages of instruments, maps, etc., exceed this in value, but according to the instructions it is not allowable to pay excess express charges, and therefore the valuation should be given as "not exceeding $50."

PRINTING.

The Assistant Comptroller of the Treasury, on October 21, 1903, decided that the Secretary of the Interior is authorized under section 10 of the act of June 17, 1902 (32 Stat., 390), known as the reclamation act, to make rules and regulations to govern the manner in which the printing necessary to carry into effect the provisions of this act shall be done, subject to the limitations of the act of January 12, 1895, and that if he should prescribe by rule and regulation that an officer doing field work should procure for his use the printing of notices and specifications outside of the Government Printing Office, as the exigencies of the service required them, the printing so done would not come within the provisions of the act of January 12, 1895, as applied to printing done for an Executive Department. If printing for such purpose can be held to be for an Executive Department within the meaning of the act of January 12, 1895, such action of the Secretary of the Interior would place it in the class of printing "otherwise provided for by law." (4 Comp., 274.) The decision concludes:

> I am of the opinion, therefore, that you are authorized to prescribe by rule and regulation the manner in which the printing described in your communication shall be procured, and that the printing so provided for can be paid for out of the "reclamation fund."

The usual or routine printing for the reclamation service is done at Washington on requisition through Departmental channels. When it is apparent that on account of an exigency printing should be done in the field, a request must be made to the chief engineer for suitable authority to be had for the general or specific purpose. Requests for such authority should state clearly the character of the printing, number of copies, character and extent of the work, and the estimated cost. No printing should be undertaken until such authorization is obtained.

LAWS COVERING PURCHASES.

As work progresses from preliminary reconnaissances to detailed surveys and examinations, a larger and larger number of articles must be purchased and a greater variety of machinery and appliances obtained, such as diamond drills and other tools of various kinds. The regulations and instructions of the Geological Survey and the customs growing out of a quarter of a century of field work govern the usual expenditures for surveys and examinations, but the additional duties of the reclamation service pass somewhat beyond the precedents established, and for this reason especial attention must be paid to the laws governing purchases in general and care must be taken to observe all such laws.

The following quotations from the Revised Statutes contain some of the most important provisions of law regarding purchases:

SEC. 3709. All purchases and contracts for supplies or services in any of the departments of the Government, except for personal services, shall be made by advertising a sufficient time previously for proposals respecting the same when the public exigencies do not require the immediate delivery of the articles or performance of the service. When immediate delivery or performance is required by the public exigency the articles or service required may be procured by open purchase or contract at the places and in the manner in which such articles are usually bought and sold, or such services engaged, between individuals.

SEC. 3710. Whenever proposals for supplies have been solicited the parties responding to such solicitation shall be duly notified of the time and place of opening the bids and be permitted to be present either in person or by attorney, and a record of each bid shall then and there be made.

SEC. 3737. No contract or order or any interest therein shall be transferred by the party to whom such contract or order is given to any other party, and any such transfer shall cause the annulment of the contract or order transferred so far as the United States are concerned. All rights of action, however, for any breach of such contract by the contracting parties are reserved to the United States.

SEC. 3739. No Member of or Delegate to Congress shall directly or indirectly, himself or by any other person in trust for him, or for his use or benefit, or on his account, undertake, execute, hold, or enjoy, in whole or in part, any contract or agreement made or entered into in behalf of the United States by any officer or person authorized to make contracts on behalf of the United States. Every person who violates this section shall be deemed guilty of a misdemeanor and shall be fined three thousand dollars. All contracts or agreements made in violation of this section shall be void; and whenever any sum of money is advanced on the part of the United States in consideration of any such contract or agreement, it shall be forthwith repaid; and in case of refusal or delay to repay the same, when demanded, by the proper officer of the Department under whose authority such contract or agreement shall have been made or entered into, every person so refusing or delaying, together with his surety or sureties, shall be forthwith prosecuted at law for the recovery of any such sum of money so advanced.

SEC. 3740. Nothing contained in the preceding section shall extend, or be construed to extend, to any contract or agreement made or entered into or accepted by any incorporated company, where such contract or agreement is made for the general benefit of such incorporation or company, nor to the purchase or sale of bills of exchange or other property by any Member of (or Delegate to) Congress where the

same are ready for delivery and payment therefor is made at the time of making or entering into the contract or agreement.

SEC. 3741. In every such contract or agreement to be made or entered into, or accepted by or on behalf of the United States, there shall be inserted an express condition that no Member of (or Delegate to) Congress shall be admitted to any share or part of such contract or agreement or to any benefit to arise thereupon.

SEC. 3742. Every officer who, on behalf of the United States, directly or indirectly makes or enters into any contract, bargain, or agreement in writing or otherwise, other than such as are hereinbefore excepted, with any Member of (or Delegate to) Congress, shall be deemed guilty of a misdemeanor and shall be fined three thousand dollars.

SEC. 3743. All contracts to be made, by virtue of any law, and requiring the advance of money, or in any manner connected with the settlement of public accounts, shall be deposited promptly in the offices of the auditors of the Treasury according to the nature of the contracts: *Provided*, That this section shall not apply to the existing laws in regard to the contingent funds of Congress (as amended July 31, 1894).

SEC. 3744. It shall be the duty of the Secretary of War, of the Secretary of the Navy, and of the Secretary of the Interior to cause and require every contract made by them severally on behalf of the Government, or by their officers under them appointed to make such contracts, to be reduced to writing and signed by the contracting parties, with their names at the end thereof, a copy of which shall be filed by the officer making and signing the contract in the returns office of the Department of the Interior as soon after the contract is made as possible and within thirty-six days, together with all bids, offers, and proposals to him made by persons to obtain the same, and with a copy of any advertisement he may have published inviting bids, offers, or proposals for the same. All the copies and papers in relation to each contract shall be attached together by a ribbon and seal and marked by numbers in regular order, according to the number of papers composing the whole return.

SEC. 3745. It shall be the further duty of the officer, before making his return according to the preceding section, to affix to the same his affidavit, in the following form, sworn to before some magistrate having authority to administer oaths: "I do solemnly swear (or affirm) that the copy of contract hereto annexed is an exact copy of a contract made by me personally with ———; that I made the same fairly, without any benefit or advantage to myself, or allowing any such benefit or advantage corruptly to the said ———, or any other person; and that the papers accompanying include all those relating to the said contract, as required by the statute in such case made and provided."

SEC. 3746. Every officer who makes any contract, and fails or neglects to make return of the same, according to the provisions of the two preceding sections, unless from unavoidable accident or causes not within his control, shall be deemed guilty of a misdemeanor, and shall be fined not less than one hundred dollars nor more than five hundred, and imprisoned not more than six months.

SEC. 3747. It shall be the duty of the Secretary of War, of the Secretary of the Navy, and of the Secretary of the Interior to furnish every officer appointed by them with authority to make contracts on behalf of the Government with a printed letter of instructions, setting forth the duties of such officer, under the two preceding sections, and also to furnish therewith forms, printed in blank, of contracts to be made, and the affidavit of returns required to be affixed thereto, so that all the instruments may be as nearly uniform as possible.

PURCHASES IN OPEN MARKET.

The most important feature as regards the current operations of the reclamation service is that contained in section 3709 (given on p. 67), to the effect that when immediate delivery or performance is required by the public exigency, the articles or services required may be secured by open purchase or contract at the place and in the manner in which such articles are usually bought and sold or services engaged between individuals.

As a rule, at present the operations of the reclamation service are what may properly be termed a series of exigencies, as distinguished between the ordinary routine of Departmental practice. The law evidently contemplates the purchase of supplies and material for well-established bureaus which are carrying on year after year operations which have been defined by law or by custom and are of such character that it is possible to anticipate months in advance what material may be needed.

The work of the reclamation service is, and must be for several years, largely devoid of routine features; the requirements are novel and future conditions unknown. The work leading up to the building of large irrigation works is a chain of events, one linked with the other in such a way that it must be finished before the next can be touched. It is impossible to look far into the future, and the completion of one detail reveals entirely unanticipated conditions in the next. If it were required for each step of the proceeding that the material needed must be obtained by advertisement and competitive bidding, months would be required to perform some of the operations which, under ordinary business practice, can be accomplished in a few days or weeks. It is obvious that the letter and spirit of the law do not compel any dilatory practice.

Under the existing regulations and customs of the Geological Survey, which govern also the reclamation service, necessary supplies and material may be purchased in the field. In the conduct of surveys and examinations in unsettled regions, where exigencies are continually arising, such procedure is essential, but as the work advances to the stage of construction and large purchases must be made, these should be fully and carefully guarded.

A somewhat arbitrary line must be drawn between preliminary surveys and examinations and work that partakes of the nature of construction. In all expenditures for such surveys and examinations, the general authority granted in advance and the approval of plans conveys authority for making purchase of animals, tents, instruments, cooking utensils, and other material which can not be obtained upon requisition.

As soon as the examinations have proceeded to a point where recommendations can be made for preliminary or final construction, the

results are presented to the Secretary of the Interior with recommendations. His approval contains or carries with it specific or general authority for making such other purchases as may be necessary, the articles to be obtained either by advertisement or purchased in open market, according to the nature of the material and the exigencies existing.

The district or local engineer, in making his plans for field work or for construction, should anticipate as far as possible not only the number of men that it may be necessary to procure through civil-service rules, but also the character and amount of purchases to be made. The plans comprising these and all other items should be prepared and submitted so far in advance of the need that they can be approved in ample time to enable advertisement to be made for material, which should be procured through competitive bids; or authority should be had covering the purchases that must be made in open market.

Even with the exercise of all reasonable foresight contingencies will arise where articles must be had without waiting to obtain authority, and it is expected that these articles will be purchased in open market and that the exigency will be plainly stated.

Responsible officers in the field must bear in mind the conditions existing and must guard on the one hand against the appearance of reckless or thoughtless expenditure and on the other against delays consequent upon requests made for authority in minor or unimportant matters. Work should be pushed forward rapidly and economically and at the same time every effort should be made to comply with the letter and spirit of the law regarding purchases made by the Government.

Purchases may be made in open market without advertising at the places where such articles as may be wanted are usually bought and sold, and in the mode in which such purchases are ordinarily made. All purchases of subsistence supplies must be limited to articles comprising the established survey ration.

In unusual open-market purchases by an agent of the reclamation service involving any considerable expenditure of money, informal proposals should be invited from the principal dealers in the articles needed who may be within his reach. All such proposals must be forwarded to the chief engineer as evidence that the prices paid were the lowest and the terms and conditions the most favorable that could be obtained.

Specific authority from the chief engineer must be obtained for purchases of an unusual character before purchase is made, except in cases of special exigency, when the absolute necessities of the service will not permit the delay incident to obtaining such authority. In such cases purchases may be made before authority is obtained, but only to the extent of relieving the immediate necessity.

District engineers are not the sole judges of the exigency spoken of in the preceding section; a full report of the facts attending purchases made without authority, accompanied by an itemized list showing articles purchased and prices paid, must be immediately submitted for the consideration of the chief engineer. With proper care and foresight such exigencies will seldom arise. Engineers making such purchases without previous authority do so at their own risk. If such purchases do not meet with approval the person making them will be compelled to pay for the articles out of his own private funds.

In submitting requests for authority to purchase, engineers must state explicitly whether or not the prices quoted include the cost of transporting the articles required to the place where they are required. If not, the estimated cost of such transportation must be stated.

Requests for authority to make purchases or for other purposes should, in all possible cases, be made by letter and not by telegram.

Engineers and bonded officers are reminded that the law does not regard open-market purchases with favor, but permits such purchases to be made only because of conditions and emergencies which inevitably arise in the service and which can not be foreseen.

A requisition for authority to make purchases in open market should, regardless of the aggregate cost, cover all the actual requirements of the service for the ensuing season or year as far as it may be practicable for intelligent business foresight to anticipate them.

RULES GOVERNING CONTRACTS.

The following rules, somewhat modified from those in use by other bureaus of the Department of the Interior, are presented for the information and guidance of the officers of the reclamation service. After full consideration and test of their applicability they will be presented to the Secretary for his final approval.

1. All contracts for supplies or services shall be made only after due public advertising for proposals, except in the case of:

(*a*) Personal services.

(*b*) Supplies not exceeding $1,000 in value at any one time.

(*c*) Goods and supplies to be purchased in open market.

(*d*) Funds allotted for construction of ditches and other works for irrigating, which may be expended in open market, under general authority from the Secretary of the Interior.

(*e*) Purchases from Indians.

(*f*) Exigency, when, in the discretion of the Secretary of the Interior, he may direct that purchases may be made in open market.

2. Advertisements for bids or proposals for construction or supplies

shall be published either in one or more papers of the largest circulation in the localities most favorable for securing the services or purchasing the articles required, or such advertisement may be by sending out circular letters and by a public posting of such circular letter in the same localities. Authority for advertising in either form must be obtained through the chief engineer.

3. Information in detail in regard to supplies or construction for which proposals have been invited should be furnished to all persons desiring it who may make application to the source designated in the advertisement.

4. Proposals should be made in strict accordance with the terms of the advertisement or circular of instructions.

5. Proposals for supplies should be so framed that each article or item may be considered separately.

6. Specifications need not be incorporated in the body of the proposal; their attachment to the proposal and the declaration that they form a part of it will be sufficient.

7. Each bid must contain the names of all parties to it or that may be interested in it, and must be signed by the bidder submitting the same with his usual signature in full

8. Bidders must designate their places of business and post-office addresses.

9. A copy of the advertisement inviting the proposal must be attached to each bid.

10. Proposals must be inclosed in envelopes, sealed and indorsed, and addressed as required by the advertisement, and must be received by the officer to whom they are addressed before the time appointed for the opening.

11. All proposals received prior to the advertised time of opening must be securely kept by the officer to whom addressed, and must not be opened nor read until the time appointed.

12. Bids shall be opened and read aloud at the time and place designated, in the presence of bidders and others who may wish to attend, and the contracts shall be awarded to the lowest responsible bidder if such award shall appear to be for the best interests of the service; but no work shall be performed in pursuance of such award, nor shall any supplies be received under it, until after the contract shall have been approved by the chief engineer and then by the Secretary of the Interior.

13. The contracting officer is at liberty to reject a bid from any person who has in any respect defaulted in any previous bid or contract.

14. Any or all bids, or any part of any bid, may be rejected if such rejection is deemed best for the interests of the Government.

15. Where samples accompany the bids, and because of the superi-

ority of the samples, or for other good cause, the contract is awarded to a bidder not the lowest, the reason for such award must be fully and clearly stated.

16. Whenever bids for any article contain conditions detrimental to the Government, they may be rejected, and the articles specified in such bids may be purchased in open market, at prices not to exceed those of the lowest bidder and not to exceed the market price of the articles, until such time as satisfactory bids can be obtained, for which immediate advertisement shall be made. Authority for such open-market purchase should be asked for, if desired, when all the bids are recommended for rejection.

17. Bids for goods to be furnished or work to be performed must be accompanied by a certified check or draft for 5 per cent of the amount of the bid, payable to the order of the Secretary of the Interior.

18. In case a bidder to whom a contract is awarded fails to execute a written contract, the amount deposited by him shall be forfeited and paid into the Treasury of the United States.

19. After a contract is duly executed the amount deposited by the bidder will be returned to him.

20. Bidders for goods and supplies will be required to furnish samples of the articles bid for in all cases, except such as lumber, building material, machines, engines, and like bulky articles, in which case the kind, size, capacity, and quality proposed to be furnished must be fully stated, the Department reserving the right to call for samples if that course shall be deemed to be for the best interests of the service.

21. When samples are required, they should be delivered at the place and before the time of opening bids, so marked that they may be identified at any time.

22. Samples must not be opened until after all bids have been publicly read.

23. As soon as practicable after the reading of the bids the samples offered should be examined and passed upon either by the contracting officer himself or by some competent, disinterested person or persons to be selected by him.

24. In making awards under bids or proposals for furnishing goods and supplies the right will be reserved to increase or diminish to any extent the quantity of any of the articles specified in any bid, and the further right to increase or decrease the quantities specified in any contract to an extent to be specified in the contract. Parties desiring to limit the quantity of any article or material offered must state in their bids the quantity proposed to be furnished, which should be followed by the word "only," and the right will be reserved to accept any part thereof, not exceeding the amount designated.

25. As soon as practicable after proposals shall have been opened

and read an abstract of them shall be made, upon which shall be entered every proposal received, the entry to comprise the names of the bidders, the items covered by the bids, and the prices at which it is proposed to supply the material or labor specified. The accepted bid, with the quantity and price of the materials or labor to be furnished thereunder, shall also be noted on the abstract.

26. An abstract of the proposals must accompany the draft of the contract, and this should be sent to the chief engineer with all other papers including the envelopes in which the bids were received, properly marked for identification.

27. When bids are opened, and it is contemplated to enter into formal written contract, with bonds, the abstracts of the bids, except in cases of emergency contracts, will be forwarded to the office of the chief engineer with recommendation for action in regard to accepting or rejecting the bid or bids received, and no such bid should be accepted or rejected by the officer in charge of work in advance of approval by the chief engineer.

28. In case a contract is made by an agent or officer of the reclamation service the abstract of proposals must be duly certified to by such agent and forwarded to the office of the chief engineer, with the original of the contract and all the papers relating to the bids.

29. All contracts made by officers of the reclamation service or agents in charge of works, as well as all papers connected with contracts, will be prepared in accordance with the printed forms prescribed by the Secretary of the Interior; and these forms and other printed forms to be used will be furnished from the office of the chief engineer when required.

ADVERTISING.

When it is intended to have any work performed, or purchase made, by contract, the specifications, except in case of an exigency contract, must be previously submitted, through the chief engineer, to the Secretary of the Interior for his approval.

In advertising for proposals for contract work to be done under the direction of the chief engineer, officers and agents of the reclamation service will state in the specifications what proportion of the total consideration of the contract will determine the amount of the bond to be required of the contractor. The Department usually requires a bond of 50 per cent of the contract price.

The law governing advertising is specific, and should be carefully observed. The following paragraphs give the principal features of the law and the current practice of the Department:

No advertisement, notice, or proposal for any Executive Department of the Government, or for any Bureau thereof, or for any office therewith connected, shall be

published in any newspaper whatever, except in pursuance of a written authority for such publication from the head of such Department; and no bill for any such advertising, or publication, shall be paid, unless there be presented with such bill a copy of such written authority. (Section 3828, Revised Statutes, United States.)

That hereafter all advertisements, notices, proposals for contracts, and all forms of advertising required by law for the several Departments of the Government, may be paid for at a price not to exceed the commercial rate charged to private individuals, with the usual discounts; such rates to be ascertained from the sworn statements to be furnished by the proprietors or publishers of the newspapers proposing so to advertise. (Act June 20, 1878.)

Blanks upon which to render the sworn statements required by the above law are furnished by the Departments upon application.

Newspapers publishing advertisements pertaining to the Executive Departments are required to forward to the Department from which the advertisements emanate a sworn statement of the commercial rates charged by them to private individuals, with their usual discounts. Changes in advertising rates take effect from the date upon which the sworn statement thereof is received at the Department, unless a subsequent date is fixed therein by the publishers. These statements will give the size of type used in the advertisements and the line rate for the first and subsequent insertions.

The Departments insist upon the *line rates*, as they furnish fewer opportunities for mistakes and misunderstandings in the settlement of bills than do rates based upon the square, inch, or folio. If the commercial rates of a paper are by the square, inch, or folio, the Government line rate should be relatively the same, or less, because, under the law, rates in excess of the commercial rates charged to private individuals can not be allowed by the Government.

The sworn statement of rates must be accompanied by a copy of the printed card rates charged to private individuals.

"Special," "local," or "reading" rates can not be allowed, and in auditing bills no allowance will be made for *displayed* or *leaded* advertisements or *prominent headings*. The matter must be set up *solid*, without paragraphing, and should be in the *type specified in the sworn statement* of rates on file in the Departments. If other type be used, no allowance will be made for additional space on that account. The number of solid lines only in advertisements, by actual count, will be allowed. All abbreviations in copy must be strictly followed. The particular column or page of the paper in which advertisements appear is left entirely to the option of the publisher.

The filing of a statement of rates in one Executive Department can not be construed as a notice to another Department that rates have been filed. Such statement must be furnished to each Department advertising.

The following is a sample of advertisement set up in accordance with the Government requirements:

> DEPARTMENT OF THE INTERIOR, United States Geological Survey, Washington, D. C., October 9, 1903. Sealed proposals, in duplicate, will be received until 2 o'clock p. m., Tuesday, November 3, 1903, at the office of the Reclamation Service, Denver, Colorado, and will then be opened in the presence of such bidders as may attend, for the construction and equipment of a metallic-circuit telephone line from the Arizona dam on Salt River to a point on that stream near Livingston, Arizona, via Mormon Flat and Fish Creek, a distance of about 56 miles. Specifications and particulars may be obtained on application to H. A. Storrs, Chamber of Commerce, Denver, Colorado, or Arthur P. Davis, Phoenix, Arizona, at whose offices the plans may be inspected. Each bid must be accompanied by a certified check payable to the order of the Secretary of the Interior for 5 per cent of the aggregate amount thereof as a guaranty that the bidder will, if successful, promptly execute a satisfactory contract and furnish bond in the sum of 50 per cent of the contract price for the faithful performance of the work. The right is reserved to reject any and all bids, to waive technical defects, and to accept any part of any bid and reject the other part, if the interests of the Government require it. Proposals should be addressed to H. A. Storrs, Chamber of Commerce, Denver, Colorado.
>
> H. C. RIZER, *Acting Director.*

An advertisement must appear in the language of the paper in which it is inserted, i. e., if inserted in a German paper, it must be in German.

In no case must an advertisement extend beyond the morning of the date therein named for the opening of the bids, sale of material, public hearings, etc.; and unless the authority given should specifically include insertion in the Sunday edition, publication in that edition must not be made.

Changes in rates or ownership of newspapers should promptly be reported to the Departments.

All accounts for official advertising pertaining to the Executive Departments must be audited in strict conformity to the foregoing provisions, and all amounts charged in excess of those allowable under those provisions will be disallowed.

Every voucher for official advertising must be rendered upon the Government official form, and be accompanied by the letter of authority to publish and a copy of each issue of the paper in which the advertisement appeared.

The permanency of the records of the Executive Departments of the Government being of the first importance, no deleble or fugitive

inks, pencils, or stamps are permitted to be used in making them up. *Therefore all vouchers for advertising must be made out and receipted with durable ink.*

The letter of application for authority to advertise for proposals should be forwarded through the office of the chief engineer; and with it should be transmitted a copy of the proposed specifications with the proposed form of advertisement prefixed.

The notices to the newspapers are signed by the Secretary and sent out by the Department in the following form:

To the Publisher of "The.....

.....

SIR: You are hereby authorized to publish the inclosed advertisement for proposals for .....

.....

to be set solid without any display in the heading, in the.....edition of your paper,.....times,.....

.....

provided your charges for the same do not exceed the sworn rates on file or to be filed in this Department, and such rates are not in excess of the *commercial rates charged to private individuals*, with the usual discounts.

You will please send one copy of each issue of the paper containing the advertisement to....., in one package, with the title of the advertisement marked on the outside. This letter must accompany your bill when presented for payment; the latter to be rendered upon the voucher inclosed herewith, and be receipted *in duplicate* by the person authorized to receipt for the money.

Respectfully,

.....

Secretary.

All matters connected with payments for advertisements should be left to the Department for attention. It is sufficient to furnish the Department in advance with the names of newspapers in which the advertisement is to be inserted and the principal facts to be stated in the advertisement, such as the time and place of opening bids, the name of the person to whom application should be made for necessary blanks, as well as all other items that may be essential to the full understanding of the matter.

If delay will be involved by correspondence, a suitable telegram can be sent, preferably at night rates, reciting these facts. The telegram should be addressed in the usual form, "Hydrographer, Geological Survey, Washington, D. C.," and will receive immediate attention.

Advertisements for other purposes, as for the recovery of lost property or strayed animals, should be made in the same manner, a request being sent by wire, if necessary, for the insertion of advertisements in specified newspapers and for a given length of time.

Section 3709 of the Revised Statutes requires that advertisements should be made for proposals, but the term "advertising" or "advertisement" is not defined in the statutes. Question has arisen as to

the nature of the advertising necessary, and it has been determined that in general this should be by proper notice inserted in newspapers, as described in preceding pages, but that in exceptional cases circulars will suffice as compliance with the law. On this point the following decisions have been rendered by the Comptroller of the Treasury (3 Comp., 175):

The question as to what is sufficient advertising, it appears to me, depends primarily upon the character of the purchase to be made or the service to be performed, the purpose of the statute requiring advertisement being evident. Section 3709 was enacted for the purpose of giving publicity and a consequent opportunity for free competition in making contracts for supplies for the Government.

"The law requiring advertisements and proposals for public contracts obviously was intended by Congress to invite competition among bidders and to prevent favoritism and fraudulent combinations in awarding contracts." (Harvey *v.* United States, 8 C. Cls. R., 501; 2 Opin. A. G., 259; 15 Opin. A. G., 538.)

Accordingly such steps must be taken as will bring to the individuals or corporations engaged in a particular kind of business, and to the public generally, knowledge of the intended purchase, and in such a manner as is designed to attract general attention. This need not necessarily be by advertisement in the newspapers, although the term "advertisement" has come ordinarily and in common parlance to refer to that form of advertisement. Bouvier defines advertisement as "a notice published either in handbills or in a newspaper" (edition of 1848); while the definition in Black's Law Dictionary is—

"Notice given in a manner designed to attract public attention; information communicated to the public, or to an individual concerned, by means of handbills or the newspaper."

* * * * * * *

After an exhaustive search of the authorities, I am unable to find in any case a judicial definition of this word, but those above given seem to have been uniformly accepted.

After referring to several cases where the word has been used by a court or Department as including posters and circulars, the Comptroller adds:

I therefore conclude that in connection with the contract in question the advertisement need not necessarily be by publication in the newspapers, but that circulars sent to the various people engaged in laundering in the various cities where such work is to be performed for the Marine Corps, together with a public posting of handbills at each of these places, announcing the intention of the Department to enter into such contracts, would meet the requirements of section 3709.

The principle above announced was reaffirmed in 3 Comp., 470.

In 15 A. G., 226, the Attorney-General held that the limitation fixed in an advertisement made by the Post-Office Department for proposals for furnishing postage stamps, restricting the proposals to "steel-plate engravers and plate printers," was compliant with section 3709, the Attorney-General saying (227):

In my opinion such limitation was proper, if it was intended to confine an answer to the proposals to such persons only as could satisfactorily furnish the article of supply needed. *The manner of advertising is left by the law to the discretion of the Depart-*

ment advertising. No particular form is required. * * * If, knowing the articles needed, and knowing that they can be supplied only by particular classes of people, he sees fit to limit his advertisement to them he may properly do so.

To the same effect is 1 Comp., 363.

In 15 Comp. (MS.), 1535, it was held that where articles of furniture in ordinary use are required, the selection of particular patterns, which can be supplied by only one manufacturer, does not authorize their purchase without advertising for proposals.

On the other hand, in 2 Comp., 632, it was held that section 3709 does not require advertising for proposals for the purchase of patented or copyrighted articles where the benefit of competition can not be secured.

In Schneider *v.* United States (19 C. Cls. R., 547) it was held that where the advertisement called for sandstone for a public building, an alteration in the contract, substituting marble at a price different from that named in the first contract (the material being the sole subject-matter of either agreement), was not a mere modification of the first, but required a new advertisement. So it was held in 20 A. G., 496, that a bid prescribing a time for completion some months later than that prescribed by the advertisement could not be accepted without new advertisement, as such acceptance would violate section 3709 and the river and harbor act of 1888. In 13 A. G., 174, it was held that the Postmaster-General had no power to renew or extend the term of a contract for printing envelopes without readvertising, as such extension would be a new contract. But a reasonable provision may be inserted in a contract whereby it may be modified from time to time by agreement of the parties, as the public interests may require, and the subsequent modification in accordance with such provision is not of the nature of a new contract, such as must be preceded by a new advertisement under section 3709 (21 A. G.,207).

From the foregoing it would appear—

(1) That the selection of a particular pattern of a common article does not dispense with the necessity of advertising, even if that pattern is manufactured by only one person or company; though the rule is otherwise where a patented or copyrighted article made by only one manufacturer is required.

(2) Where advertisement is necessary, it may be either by publication in a newspaper or by circulars and posters. Whether circulars alone would suffice does not appear to have been decided; doubtless it would in some cases.

(3) No particular form of advertisement is necessary.

(4) The advertisement may limit proposals to the particular class of persons qualified to supply the articles in each case.

(5) The advertisement may reserve the right to reject any and all bids and this right may be taken (14 A. G., 682).

(6) The advertisement must describe the subject-matter of the proposed contract with a reasonable degree of accuracy and certainty. Thus if the advertisement is for delivery of material only, a different material can not be substituted in the contract, nor can a contract for material and services be substituted (15 A. G., 253). The time for completion stated in the advertisement can not be extended by the contract, though extension may be made by agreement after part performance; nor can a new contract for similar supplies be made after performance of the first one without a new advertisement.

(7) It is, however, entirely proper that the contract provide for such modifications as may become necessary from time to time in the interest of the public service.

SPECIFICATIONS AND PROPOSALS.

In all cases where advertisements are to be issued inviting proposals for material or labor, for works of repair or construction, or other like works of the reclamation service, it is required that full and detailed specifications be prepared and printed in time to supply all parties desiring to bid under the advertisement.

The specifications should be comprehensive, clear, and sufficiently detailed to enable bidders to understand fully what is required; and as it is designed that these specifications, together with the advertisement, be attached to and form a part of the contract, it is important that great care be observed in their preparation in order that all questions that may arise during the execution of the contract may be settled and determined by reference to the instrument itself. A copy of the advertisement should be printed with and form a prefix to the specifications.

A general form of specifications available for most cases has been prepared. It may be necessary for some classes of work to make changes or additions, but in nearly all cases this form can be used as the first part of the specifications, covering the general conditions of building and the relations between the contractor and the Government. The particular requirements of the work form the subject of additional paragraphs. In all cases where drawings are to accompany the specifications it will be necessary to reproduce a large number of them, for at least five copies of each contract, with copies of the accompanying specifications, will be required, and other copies will be needed for filing, reference, and other purposes. The photographic laboratory of the Survey can make these reproductions rapidly and at small expense from tracings or black and white drawings. As soon as the tracings or drawings which are to form part of the specifications

have been prepared in final shape, they should be forwarded to the chief engineer for duplication. The specifications should contain a list of drawings, each described by number and title.

CONTRACTS AND BONDS.

All contracts to be executed under the reclamation law must be made in the name of the Secretary of the Interior upon the general form used by the reclamation service. The contracts should be accompanied by bond of conventional form. The form of the contract and bond are shown on pages 110–114.

It is customary to execute 5 copies of the contract. Only one copy of the bond need be executed. A copy of the contract is ultimately sent to each of the following places:

(*a*) To the Treasury Department.

(*b*) To the Department of the Interior for the files of the returns clerk.

(*c*) To the contractor.

(*d*) To the engineer in the field.

(*e*) To the chief engineer of the reclamation service, in the office of the United States Geological Survey.

The Treasury Department retains its copy of the contract in order that payments can be indorsed upon it and a record thus kept of these until the contract is completed.

The returns clerk of the Department of the Interior keeps a complete file of all contracts made by the Government, as required by law. In this office are to be filed not only the contract when executed, but all of the papers relating to the contract, including the rejected bids and copies of all correspondence explaining the award to the successful bidder. Care must be taken to preserve all rejected bids and file them with the accepted contract.

The contractor, of course, requires a copy of the contract, and the engineer in the field should have one. The chief engineer, being the certifying officer, should also have a copy for reference in his office. Thus it is desirable to have five complete copies.

In making the returns to the Department care must be taken that all of the papers are carefully kept together and so marked as to be understood. The extract from the Revised Statutes, noted on page 68, gives in detail the requirements of the returns office (sec. 3744 to sec. 3747).

Whenever the contract for materials or work can be let in separate divisions or classes, each part which might be contracted for separately should be listed in the specifications under a separate heading, as Division or Class 1, 2, 3, etc. When possible, the detail specifications

relating to each division or class should be so separated that the portion of the specifications relating to that division or class will be complete in itself and not require the inclusion in the contract of details relating to another division or class. So also a reference to Class 1 or Division 1 in the contract should be sufficient to fix its scope exactly.

Regulations to govern the execution of contracts and bonds were issued by the Secretary of the Interior under date of April 8, 1903, see page 114.

EIGHT-HOUR DAY.

The law in regard to the eight-hour day, so far as it relates to the reclamation service, is as follows:

Provided, That in all construction work, eight hours shall constitute a day's work, and no Mongolian labor shall be employed thereon. Section 4 of the reclamation act approved June 17, 1902 (32 Stat. L., 388).[a]

Eight hours shall constitute a day's work for all laborers, workmen, and mechanics who may be employed by or on behalf of the Government of the United States. Section 3738 United States Revised Statutes (act June 25, 1868, 15 Stat. L., 77).

Be it enacted, etc., That the service and employment of all laborers and mechanics who are now or may hereafter be employed by the Government of the United States, by the District of Columbia, or by any contractor or subcontractor upon any of the public works of the United States or of the said District of Columbia, is hereby limited and restricted to eight hours in any one calendar day, and it shall be unlawful for any officer of the United States Government or of the District of Columbia or any such contractor or subcontractor whose duty it shall be to employ, direct, or control the services of such laborers or mechanics to require or permit any such laborer or mechanic to work more than eight hours in any calendar day except in case of extraordinary emergency.

SEC. 2. That any officer or agent of the Government of the United States or of the District of Columbia, or any contractor or subcontractor whose duty it shall be to employ, direct, or control any laborer or mechanic employed upon any of the public works of the United States or of the District of Columbia who shall intentionally violate any provision of this act, shall be deemed guilty of a misdemeanor, and for each and every such offense shall, upon conviction, be punished by a fine not to exceed one thousand dollars, or by imprisonment for not more than six months, or by both such fine and imprisonment, in the discretion of the court having jurisdiction thereof. Sections 1 and 2, act of August 1, 1892 (27 Stat. L., 340).

It will be observed that the law applies only to laborers, workmen, and mechanics employed in the construction of public works. The Comptroller of the Treasury has made a specific ruling to the effect that one who is employed under the designation of "special laborer" is not included in this description, the reasoning of his decision being that it applies specifically to mechanics and those employed in ordinary forms of labor. (6 Comp., 180.) Accordingly, cooks, field assistants, chain men, instrument men, etc., would not be subject to the provisions of this law.

The enforcement of this law as between the contractor and his employee has been held not a matter for departmental action.

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[a] Pp. 259–261 of this paper.

OUTFITTING.

At the present time much of the work of the reclamation service consists of preliminary surveys, and hence many new equipments are being and must be procured in the near future. In obtaining the various articles that may be required, care should be taken that all needs are provided for and at the same time that no unnecessary material is purchased. Instruments, stationery, and the lighter articles can be procured from the Washington office. Requisitions for these should be made as far in advance as possible. Attempts should be made to keep a record of all instruments and of their condition, as well as of the degree of use that is made of them, so that plane-table outfits, transits, levels, etc., may be transferred quickly, when not in use, to points where parties are preparing to begin work.

Horses, mules, and wagons can usually be purchased or hired near the point of work. It is preferable to procure such means of transportation, if suitable arrangements can be made, by hiring a man and his team by the day or month. The man is thus responsible for the horses and wagon and for keeping the wagon in repair, as well as for seeing that the horses are properly fed, shod, and cared for. If it is impracticable to hire in this way, then teams must be purchased. Good serviceable animals and wagons should be procured, for it is usually more economical to pay a fair price for a suitable outfit than to save a few dollars at the outset and run the risk of a breakdown in the field.

The attempt is being made to set certain standards of equipment, and a report upon this subject has been prepared by members of the survey (see p. 267).

Tents may be procured upon requisition sent to the office of the chief engineer, suitable arrangements having been made with the Quartermaster-General of the Army. These tents are shipped from the nearest supply depot, and the cost is ultimately paid from the Washington office. The quality of the tents is usually good, although complaint has been received from men in the field of patched, worn, or partly mildewed tents being issued. To avoid the delay and uncertainty of obtaining tents from the Army, arrangements have been made with a manufacturer in Denver to furnish tents for definite prices. It is, therefore, possible to procure tents by direct purchase from Denver or in open market at other points whenever necessary.

Stoves, cooking utensils, various implements, and groceries should be purchased as needed from the nearest and cheapest point. The vouchers for these should be carefully itemized, the quantity and the unit price being clearly stated. The bills should be checked by the assistant engineers or aids to see that the items and additions are correct to the last cent. When so checked the fact should be stated on the margin or back of the voucher ("Checked by John Smith") to

place responsibility in case of error. Small mistakes of a few cents on large itemized bills are extremely annoying, as they delay the work of the disbursing officer and prevent prompt payment of other vouchers. For this reason, if for no other, as great care should be required in store accounts as in level or transit notes.

Attention is called to the necessity for strict economy in outfitting and in the execution and administration of all works and duties of the reclamation service. The number of assistants, draftsmen, clerks, mechanics, laborers, and all other employees should be reduced to the lowest possible limit that may be consistent with efficiency.

FIELD OFFICES.

During the time of preliminary reconnaissance and surveys leading up to construction, the district offices are located at some central point or important town. It is apparent, however, that as soon as construction begins, the district offices should be placed upon or near the works, in order to insure full oversight; also to give the younger assistants and others connected with the office as full information as possible concerning construction work and all matters pertaining to it.

Pending the time of occupancy of these field offices suitable rooms can be rented in nearby cities or towns. General authority has been granted by the Secretary for the rent of such office rooms, to be charged to the reclamation fund. In each instance the renting of the room is to be promptly reported to the Department for its information and confirmation.

When the plans have been determined upon, a convenient but relatively inexpensive office should be built and so located, if practicable, that it will serve as the permanent residence of the keeper, who will ultimately live at the headworks. Around this should be symmetrically placed other structures for the convenience of the men, and some of these may be sufficiently permanent to be left for stables or outbuildings for the keeper. While new and clean they will serve as lodging houses and furnish accommodations for the assistants employed on the work.

The field offices should be provided with ordinary cooking and dining facilities, the usual office furniture of desks, drawing tables, chairs, book and filing cases, and should also have plain, substantial sleeping accommodations for the use of the district, supervising, or consulting engineers when visiting the work. By providing these the expense of living at a hotel will be avoided. All purchases of this kind should be estimated in advance and authorized in the customary manner.

Where lumber is expensive it is practicable to purchase portable houses, of which several kinds are on the market. These can be quickly erected and moved from time to time along the line of the work. They are preferable to tents, as they are not shaken by ordi-

nary winds, are relatively free from dust, and by their use moving is expedited and the preparation of drawings and ordinary clerical work is carried on more effectively.

A commissary should be maintained for all of the men, and care taken to see that everything is neat and systematic, but not expensive. It is of the utmost importance to produce a good impression on persons visiting the works in order that they may see that everything is being conducted in a proper manner. The district engineers must bear in mind that much of the future efficiency of the service depends on creating a good opinion in the minds of the public, particularly in a farming community. If there is apparent extravagance, even in small matters, this will reflect unfavorably. If, on the other hand, public men and citizens interested who visit the work find that everything is being carried on systematically, and with due regard to economy, the service will be correspondingly strengthened.

The statutes require that not less than seven hours labor shall be performed per day by employees of the Government, and limit the time of ordinary labor to eight hours per day. District engineers are expected to fix office hours of not less than seven hours nor more than eight hours, and to arrange them in accordance with local custom and convenience. As a rule the hours should be from 8 to 12 and from 1 to 5.

The office or some other room at the headquarters should be so arranged that it will be comfortable and convenient for the assembly of the men during the evening and at other times when not employed, so that there will be reasonable inducements for the men to read, study, or discuss engineering matters. The engineers in charge should bear in mind at all times that it is necessary not only to build good works, but to improve and strengthen the men who are executing them.

It is very desirable to keep the younger assistants out of the cities, and to bring them together in fairly comfortable camps or headquarters, where they can be associated at all times with planning, designing, and constructing. Every reasonable effort should be made toward this end.

SUBSISTENCE OF MEN.

Subsistence furnished to persons employed in the reclamation service is neither an emolument nor a perquisite, but is furnished solely for the purpose of facilitating the field work of the parties, and upon this principle individual cases will be decided by the chief engineer, who may at any time make such changes as may seem most feasible or economical for the Government.

The regulations and instructions of the Geological Survey cover largely the various questions arising as to subsistence. In camp provision is made for the wants of all men employed; groceries and other

supplies are purchased in accordance with the ration list; a cook is employed, and all necessary articles are furnished for board and lodging, such as tents and dining accommodations.

The men are expected to furnish their own blankets and a cot, if one is desired; also towels and all similar articles for personal use. The Survey furnishes the cook and kitchen utensils, as well as food of good quality and in sufficient quantity for men undergoing severe physical exertion. Luxuries or unusual articles are not purchased, and medical attendance is furnished only when away from settlements or in localities where the ordinary practitioners can not be had. It is understood in engaging the services of assistants and field men that they will be supplied with ordinary and necessary medical supplies suitable for the peculiar conditions or exposure under which they are working.

Only those persons directly connected with the work are subsisted in camp. Visitors should not be entertained, except under conditions leading to the furtherance of the work. In short, every effort is made to provide comfortable and satisfactory subsistence and to avoid extravagance and unnecessary expense. Field parties, as a rule, move camp frequently and all unnecessary furniture must be excluded, as well as all persons not absolutely needed for the work in hand.

It has been the custom at the end of the field season, on the discontinuance of field work, to bring the regular employees to Washington or to some central point. Here, following usual customs, the men supply their own subsistence, usually returning to their families or permanent places of abode.

Men regularly employed at a certain point are not entitled to subsistence as are field assistants who are required to move from point to point with the camp, or shifting work. Where men are ordered to a regular office for work, or to a locality where they will practically establish a home, it is not allowable to pay for subsistence or to cover this by any form of expenditure.

As construction is taken up and the men are located at a given point for an indefinite time, it is expected that they will provide their own subsistence and establish homes, if convenient. Discretion is left largely to the district engineers in this particular, and care must be exercised by them in making recommendations covering exceptional cases.

Men who are employed for field work and paid a salary which is understood to be for this work, with subsistence furnished in the field, should, if ordered to remain at a definite point for several days or weeks, be reimbursed for reasonable and ordinary cost of board and lodging while at this point. If, however, they are ordered to a locality, with the probability of remaining there for a month or more,

they should not be allowed subsistence unless special authority is given by the chief engineer.

During the winter, or at times when field work can not be actively carried on, the men should be brought to headquarters camp and subsisted there. If they prefer to bring their families and locate them at near-by points, the men will be expected to exercise their option as to whether they personally will take a part of their meals at the general commissary. If the men prefer to remain with their families no allowance can be made for this. It should be plainly understood that a commissary is provided for the subsistence of the men, and that it is optional with them whether they take their meals at the commissary or not.

Men instructed to remain at a town or other locality for a definite time exceeding two weeks will be expected to make arrangements for their own subsistence.

It is not expected that any general rules can be formulated which will be sufficiently definite for every individual case, and such cases will be decided by the engineer in charge upon the principles above laid down. In case of doubt, or disagreement, the case will be referred to the chief engineer for decision.

In the Corps of Engineers of the United States Army conditions exist similar to those prevailing in the reclamation service. Every assistant engineer in this corps, from the highest grade to the lowest, is appointed under civil-service regulations at a specified salary per annum. These assistant engineers receive no allowance in lieu of expenses, but have all actual field expenses and subsistence paid when on field duty. The practice in connection with detail to this work is variable. There are no hard and fast rules, and each case is considered separately by the officer in charge and the Chief of Engineers. In their view the man in the field has harder work, longer hours, and more arduous duties than the man in office. Besides, he has two establishments to support, his family at home and himself in the field. If his office is at a permanent station, he has his family with him, and his living expenses are little or no additional charge upon him. Therefore under army regulations the field man is paid more than the man who is engaged in office work. This payment is made in the form of subsistence; in other woods, the man on field duty has added to his pay his subsistence. When he is engaged in office duty he receives his pay only. This is identical with the usual practice of the Geological Survey. Where assistant engineers or subassistants of the corps of engineers of the army are permanently located at Cincinnati, Frankfort, etc., they subsist themselves from the day they report for office duty. When ordered into the field they are subsisted on house boats or otherwise. In some cases men are moved

about so frequently that they may be said to have no permanent residence. They may be in Frankfort for two months, may go to the field for a time, and then be ordered elsewhere. In such cases their subsistence is paid in the temporary office as well as in the field. Where, however, a man is detailed to an office for several months, and is so notified, it is assumed that he will make such necessary permanent arrangements concerning his living as may seem desirable.

PROPERTY.

In the operations of the reclamation service property is acquired in various ways and must be accounted for with exactness. The system in use by the Geological Survey is being modified and the attempt made to simplify the making of property returns.

The Regulations of the Geological Survey, beginning on page 40, gives most of the essential facts concerning property, and the Instructions, on page 38, relates to the matter. The most important fact to be noted is that property acquired by purchase, transfer, etc., is divided into two classes, expendable and nonexpendable.

Expendable property, such as groceries and supplies of various kinds, is to be used or consumed for purposes of the service. A certificate by a responsible officer to this effect is all that is required. Care should be taken, however, to verify the fact that the expendable property has been put to proper official use.

Nonexpendable property must be classified and carried on property returns from quarter to quarter indefinitely, or until the articles are worn out or disposed of in some lawful way. Care must be taken to get rid of unnecessary articles and keep the property returns free from accumulations of things not needed.

At intervals of six months or a year all unserviceable articles should be gathered together at some convenient point and inspected by proper officers. Authority for such inspection should be procured in advance from the chief engineer, and all requirements preliminary to the proper disposal of the articles should be observed.

Auction sales should be had of all unserviceable animals and equipment, after suitable authority has been procured and advertisement made in the customary manner. Blanks for this purpose and for making returns should be obtained from the chief engineer. An auctioneer, if employed, can be paid out of the proceeds and the necessary cost deducted from the returns.

The proceeds from auction sales should be transmitted to the chief engineer, with all the papers, to be forwarded to the Department.

Each district engineer and principal officer in the field is a custodian or subcustodian for the chief engineer. Property returns must be made quarterly and must be explicit in all details, every article of nonexpendable property being accounted for in one way or another.

NEW PROJECTS.

All projects for new works must be submitted for consideration to the Secretary of the Interior through the chief engineer and the Director of the Geological Survey.

When plans for work of any kind have been sanctioned by the Secretary of the Interior or by the chief engineer, they must not be departed from without the express permission of the Secretary of the Interior or of the chief engineer.

Public works in charge of the reclamation service will be inspected at least once a year, and also when completed, by such officers as the chief engineer shall designate.

There may be one or more boards of engineers whose duty it shall be to plan or revise, as may be directed by the chief engineer from time to time, projects of reclamation required for the development of the arid lands of the United States; also to consider and report upon such matters as may be referred to it by the chief engineer (see also p. 33).

It shall also be the duty of any member or members of the board, whenever required by the chief engineer, to inspect and report upon any of the works of construction or other operations, the inspection reports to be made to the board and by it submitted to the chief engineer with such remarks as may be deemed proper.

The board shall consist of not less than three officers, to be designated by the chief engineer with the sanction of the Secretary of the Interior or of the Director of the Geological Survey.

Two members of the board shall constitute a quorum for the transaction of business, but its final decisions in important matters must be sanctioned by a majority of its members.

The reports of the board will be made to the chief engineer. It will keep a journal of its proceedings, and will on the last day of each month report to him the movements of its members and a brief statement of the occupation of the assembled board.

The chief engineer submits all important reports of the board, with his views thereon, to the Secretary of the Interior through the Director of the Geological Survey. No important work should be undertaken without the consent of the Secretary.

The orders of the senior officer of the board shall be authority for journeys made by its members and associates and for the necessary expenditures incurred by the board in pursuance of its duties, but the authority of the chief engineer must be obtained by the senior officer of the board for all journeys made by the board or members thereof to localities other than the usual place of meeting.

Questions as to water power and pumping should be brought to the notice of the electrical engineer by the engineers who encounter these

problems. The feasibility of a project may depend on the possibilities regarding power development and use, and as much time should be allowed for studying this phase of the undertaking as for the investigation of other engineering questions. If this information is withheld until all the plans for the irrigation system have been worked out, it may often happen that insufficient time will remain in which to make a comprehensive study of the design. Moreover, some data may be lacking or some modification of the approved plans of dam, canals, etc., may be required in order to secure the best design of power plant, thereby delaying the execution of the project.

Whenever the investigation of a new project has proceeded far enough to warrant a visit and examination by the consulting engineers, their report regarding the project should state whether or not any power development is contemplated. In case such development is contemplated, the report should, if possible, state briefly the amount and probable use of the power to be developed, and should also state when, in the opinion of the consulting engineers, the electrical engineer should receive from the engineer of the project such preliminary data as will enable him to study the situation sufficiently to decide what special additional information is wanted by him in working out the designs, plans, and estimates of the power system, or when, in their opinion, the electrical engineer should visit the project and consult with the local engineer regarding power possibilities.

In considering new projects it is necessary to take into account not merely the physical conditions, but many other factors. Among the most prominent of these are the questions as to whether the particular work can best be executed by private agencies. In many localities where the projects have been studied and the results generally known there are persons who come forward with a plan to build a portion of the project and who express the desire to have the reclamation service turn over to them the simpler or more attractive part of the scheme for development. There are cases where it might be better for all concerned for the Government to decline to take up the work, and full consideration must be given matters of this kind.

If the reclamation service is to step out of the way whenever a corporation or association of individuals expresses a willingness to build an irrigation project, the result will be a surrender of the best projects, leaving those which are lacking in merit. Works constructed by private corporations are built for profit, and every dollar that the settler can pay will be extracted from him.

The usual method of procedure of a private scheme is about as follows:

The agents of a new water corporation proceed through a district, seeking contracts for water to be furnished to the land, these contracts being in the nature of a first mortgage. With these contracts in hand

a promotion company then attempts the sale of bonds. These bonds must be heavily discounted in order to find purchasers and to pay commissions. They are frequently disposed of on a basis of about 75 or 80 cents on the dollar. The promotion company usually surrenders to the bond underwriters the controlling interest in the common stock in the water company. The profit of the promotion company consists in the retention usually of from 33 to 49 per cent of the common stock of the water company.

The settler has to meet under these conditions three distinct charges: First, discount on the bonds; second, interest on the entire bond issue; third, interest on the capital stock. The consequence is that the final cost is far larger for works hastily and poorly built than if the Government constructed permanent works to endure for all time.

The constructing corporation usually plans to build a headworks at the most convenient point and to reclaim only the most readily irrigable lands. This results in condemning to sterility, at least for a long period of years, the remaining less desirable or less accessible lands.

On the other hand, the arguments in favor of stepping aside for private corporations may be summarized as follows:

The object and purpose of the reclamation act is primarily the development of the arid West. It is admitted that the present funds are inadequate for complete development, and this being the case it would seem improper for the Government to stand in the way of the construction of works by private capital when the Government could go elsewhere for the construction of other works in more remote regions where private capital is unwilling to invest.

If the Government refuses to step aside in irrigation projects, it will in each case incite the vigorous animosity of persons or corporations who desire to speculate.

Again, as it is impossible for the Secretary of the Interior to make decisions in all cases until definite plans and surveys are submitted, it is unjust for him to prevent the construction of works by private capital when the promoters are certain that they want the opportunity for investment.

The arguments for and against the construction of any particular project by the reclamation service should be carefully considered and particular weight be given to conditions involving the irrigation of public lands. The reclamation of these can unquestionably be carried on better under the reclamation law than by any form of private enterprise. It must also be borne in mind that the construction of large works of this character must involve endless strife between the water user and the company. As a rule, the water companies have in the past been financial failures and have not brought about the best development of the country.

ESTIMATES OF COST.

In making estimates for any new project great care must be taken to determine accurately its probable cost and not to underestimate this. The landowners must ultimately pay the proportional cost per acre, and if the cost is found to be less than the original estimate the result will be satisfactory to all, but if the cost exceeds the estimate and the price per acre is placed above that originally fixed, the landowners may think that they have not been treated fairly. The cost per acre in any event will probably be far below the ultimate value of the water.

Engineers connected with the designing of large works must bear in mind that the greatest efficiency and economy of these works usually results from a careful study of the plans when nearly completed. In the initiatory stages of the reclamation service it has been necessary to push work rapidly in order to show results, but before the general plans have been approved and the work outlined it is highly essential for the engineers to study with great care and reasonable deliberation all of the important details and to take sufficient time to do this thoroughly.

CONSTRUCTION.

The reclamation law evidently contemplates that most of the work shall be executed by contract. Many portions, however, must be executed on what is commonly known as "force account"—that is to say, by the tools and appliances furnished by the reclamation service and by men employed by the day or by ordinary contract. There are many situations, especially in new work, where it is impracticable to furnish sufficient knowledge in advance to enable contractors to bid closely, and where the work can be done more effectively and economically by day labor. This, for example, has been found to be the case in drilling for foundations, where the locality is remote from lines of transportation and the underground conditions are uncertain. It has been found by actual experience that holes can be sunk or diamond-drill cores extracted at about a third of the cost at which the work could be let by contract.

In many operations also the work from day to day depends on the discoveries made under changing conditions. Here it is impracticable to let contracts in such form as to cover all of the contingencies, and the reclamation service must thus have a somewhat extensive plant for executing various kinds of work and must employ means utilized by other constructing organizations for pushing forward work rapidly and effectively.

AUTHORIZATIONS

It is important to bear in mind the origin of authority to execute work and expend funds, and at all times to ascertain that full authority is possessed for the proposed operations. The origin of all authority is traceable to acts of Congress. These confer, in general or specific terms, upon the Secretary of the Interior, or other executive officers, the power and duty of performing certain things in definite ways.

The act of June 17, 1902 (32 Stat. L., 388), is the foundation upon which the work of the reclamation service is established. The details of the operations are, however, modified or guided by general laws and departmental rules, which have the force of law.

The Secretary of the Interior is the responsible officer, who in turn delegates certain authority to others. This is usually general in character and covers ordinary field operations and construction. It covers the purchase of the articles customarily needed and employment of persons, but is limited in detail by certain customs or by decisions of the Comptroller or Auditor. It must not be assumed that all freedom enjoyed by an individual or a corporation is possessed by the officers of the reclamation service.

It is impossible to state the limitations which are set to general authorizations. As a rule it may be said that by the employment of sound judgment in carrying out the purposes of the reclamation service there will be little difficulty encountered, as the attempt is made to keep the work on a substantial business basis.

The engineers and certifying officers must, however, not be surprised at meeting occasional unexpected obstacles or limitations of authority, especially in minor details. These, though trying, must be patiently and loyally endured, as protest is of little avail. To illustrate one of the minor limitations may be cited the fact that in transmitting small amounts of money in payment of official debts incurred it has been found by experience that the Comptroller will not permit the payment of the fee for a post-office order, but will allow the fee for registration of 8 cents, if supported by evidence. Thus, when necessity arises for transmitting monthly payments to observers, or similar small amounts, the responsible officer can either send his own check or put the money in a registered letter, but can not expect reimbursement if a postal order is used. These minor limitations can usually be made known only by experience.

In all work it is essential to have definite and explicit authority for expenditures of every kind. At the beginning of the fiscal year the Director receives estimates of proposed expenditures, and submits these to the Secretary of the Interior for approval, general authority being by this means obtained for all expenditures. The Secretary of

the Interior, through the Director of the Geological Survey, allots to the various branches and divisions of the Survey certain sums from suitable appropriations, and gives authority to the responsible officers for expenditures in general terms.

In the hydrographic branch the hydrographer in charge, or chief engineer, makes specific allotments to the various men and these allotments must not be exceeded without further authorization. Each chief of party, district engineer, or hydrographer must be sure that this authorization is explicit and complete, and in rendering statement of expenditures from time to time should quote the original authorization, showing the date and amount of allotment.

Before any important piece of work is undertaken which is not within the allotment, specific authority for this must be had; no investigation or piece of work should be begun until estimates for it have been made and authority obtained covering the necessary expenditures.

Authority to purchase materials and execute work by contract or otherwise, in the manner most economical and advantageous to the United States, must be granted by the Secretary of the Interior in some one of various forms.

When an officer has occasion to propose the execution of any special work, as repairs, temporary buildings, special surveys, rebuilding or repairs of roads, etc., the proposition should always be accompanied by an itemized estimate, where this is practicable, of the probable cost of such work.

No sales of property will be made without authority from the chief engineer. Private sales are prohibited.

Property pertaining to one appropriation, when transferred for expenditure in connection with another, will be paid for from the latter, as in the case of other property purchased from the usual sources of supply.

Special authority for the transfer of property is necessary when the purchase of such property from the usual sources would, under the regulations, require that authority.

It is desirable to sum up briefly the sequence of events and the steps to be taken by the district engineers in the conduct of any particular piece of work from its initiation to the time of construction. At each important step certain authorization must be obtained in order to keep the Department fully informed of the work and guard against error or misunderstanding.

The first step in any project is the reconnaissance, or preliminary survey. The engineer in the field initiates this by calling it to the attention of the chief engineer; or, the latter, following a request from citizens or public bodies, refers the matter to the engineer in the field for examination and report. In any case, specific authority for mak-

ing examination should be obtained, and an estimate should be prepared as to the probable cost in which should be stated the methods to be pursued. In this preliminary examination care should be taken to ascertain the essential facts completely and at the same time not to make elaborate surveys where an inspection by a competent engineer would determine the essential points.

Upon the completion of a preliminary examination, results are reported to the chief engineer, and are summed up in certain recommendations, either that no further action be taken or that more complete surveys be made. At this time a concise statement of all of the conditions should be given and an outline of the surveys and examinations necessary, together with an estimate of the cost of these, the time probably required, and the number of men and instruments and amount of equipment necessary. Specific authority to make these surveys should be obtained from the chief engineer before notable expenditures are made on the work.

The surveys thus authorized should be conducted vigorously; reports and estimates should be prepared and submitted to the chief engineer, to be referred by him to the consulting engineers for their opinion. When this opinion is rendered, it is transmitted through the Director of the Geological Survey to the Secretary of the Interior, with recommendation for abandonment or for further work of construction.

Authority from the Secretary of the Interior for further work is usually given in general terms, and definite instructions are issued by the chief engineer, covering the necessary details. If the authorization to proceed with construction necessitates the letting of contracts, the form of advertisement should at once be prepared and sent by the district engineer to the chief engineer, to be approved and forwarded to the Department. Advertising for bids is done usually through the miscellaneous division of the Secretary's office; if, however, by special authority, advertising is done by the engineer in the field, a copy of the authority and proof of advertisement must be submitted with subsequent papers.

When bids are to be opened, suitable officers will be designated to form a board, and the chairman will be responsible for the safety and the correct form of the papers. The board will open the bids in the presence of bidders and others interested, and will note particularly whether these are complete, and whether certified checks are inclosed. After canvassing and comparing the bids the board will make a formal report to the chief engineer, stating that the bids have been opened in accordance with advertisement, and that after examining the bids all of the papers relating to the matter, including the bids recommended for rejection, are transmitted with recommendations based upon certain facts

In case all bids are recommended for rejection, or if the lowest bid is not accepted, the reasons must be clearly given, and evidence presented in such form as to stand criticism by rejected bidders or other persons. All papers and bids must be transmitted with the report.

All papers, including the envelopes in which the bids were filed, must be transmitted to the chief engineer, these being listed and properly marked for identification. The letter of transmittal containing the recommendations must be explicit, showing all of the steps taken and the authority under which each act is done.

In case the general authorization from the Secretary of the Interior for construction is of such character that material must be purchased or labor performed without advertisement for bids, the authority for so doing must be explicit, and reference must be made to this authority in rendering accounts. It is not necessary to procure specific authority for making purchases of supplies to be used in preliminary surveys and field examinations, but if in the course of construction work it becomes necessary to expend considerable sums of money—for example, over $1,000—or to make unusual purchases, separate authorization should be obtained through application to the chief engineer. In rendering the vouchers for such expenditures the authorization must be quoted.

In short, the engineer in the field must keep in constant touch with the central office, must push the work vigorously, foresee probable contingencies, secure authority for unusual matters, and at the same time not wait for authority if the exigency demands prompt action. Wherever such action is taken without the usual authority, the facts should be plainly stated in subsequent communication.

EXAMINATION OF LAND TITLES.

The examination of title to the lands involved in the various irrigation projects must be made at some time during the investigation. It is necessary before construction can begin to know as to each tract whether it is public or private land; while in connection with the purchase of right of way for canals or of lands to be flooded in reservoirs, a complete abstract of title will be required.

The first step will be to obtain a copy of the record of the local land office to show the present status of each tract. It will not, of course, be necessary to copy notes of entries that have been canceled, of contests that have been decided, or the like. This record will give the nature and number of the entry, a description of the land, the name and post-office address of the entryman, and other details. Such a record will show what land is vacant, what land is entered, and what land has been patented. A book has been prepared for making these abstracts and can be obtained on application.

The title to lands to be purchased or over which right of way is required must be carefully examined and brought down to date, in order that the office may be informed as to the present owner and the validity of his title when negotiations for the purchase are taken up.

In connection with the matter of right of way for canals to be constructed by the reclamation service, attention is called to the fact that, by decision of the Secretary of the Interior of October 5, 1893 (Decisions of the Department of the Interior Relating to Public Lands, vol. 17, p. 521), it has been held that all lands entered subsequent to October 2, 1888, are subject to a right of way for ditches or canals constructed by the authority of the United States, in pursuance of the act of August 30, 1890 (26 Stat. L., p. 391). A circular has been prepared to be used in notifying the owner of land subject to such reservation that it is proposed to construct a ditch over his land (Form 9–263). It contains a copy of the law, and is to be used only after final decision has been reached that such construction will be undertaken. It is not advisable to issue this circular prior to that time, because the reclamation service would then be committed to the construction of a canal over a particular route, and a complication would thus be introduced which might interfere with necessary negotiations for purchase of rights of way at other points along the same line.

A knowledge of the lands for which patents have issued having been obtained from the records of the local land office, it does not appear necessary to make an abstract of the title in order to carry down the transactions to the present time. When the irrigation works are completed and the reclamation service has announced that applications for water rights will be received the parties owning lands will, in making application, be required to file affidavit that they have title in fee simple to the particular tracts which they desire to water. This assertion of title can be verified by the tax records of the county at very small expense. If the person making application for water right is also shown to be assessed for taxes on the tracts in question it is fair to presume that he has title in fee simple, as shown by the face of the record. The intermediate transactions are not important for the purposes of the reclamation service. This assertion of title, under oath, and verification by the tax records would undoubtedly be sufficient to protect the Government against fraud in this direction, and the expense of tracing down the title would be thus avoided.

Negotiations for the purchase of land, rights of way, water rights, or other property should be made as representing the Secretary of the Interior, for and on behalf of the United States of America. Authority to act in this capacity will be given by the Director of the Geological Survey to the person designated to take up the work.

In connection with some of the projects it may be necessary to make a thorough examination of the water rights of record. The conditions involved in each project will determine whether such examinations are necessary and also whether other special examinations of particular tracts or claims are required for the proper consideration of the details of the project. When the engineer has made a complete study of the water right and other conditions affecting the proposed work he should make full report thereon to the chief engineer, with recommendation as to the method of dealing with them.

LAND PURCHASES.

The reclamation law, section 7, authorizes the Secretary of the Interior to acquire, if necessary, rights to property by purchase or condemnation. It is preferable to make every reasonable effort to obtain favorable terms by purchase rather than to resort to the tedious and sometimes uncertain processes of law for condemnation. The procedure for the purchase of land by the Secretary is as follows:

When the work on any project has reached the point when construction will soon be decided upon, the district engineer will take steps looking to the acquisition of rights or property needed, such as water rights, lands, rights of way, etc. Options for the purchase of such property should be obtained as much in advance of construction as possible, without committing the Government to any particular plan of operations prior to a definite decision by the Secretary of the Interior. The district engineer must exercise discretion as to the best time for beginning the work.

The district engineer or his assistant or other person designated by the chief engineer should examine and inspect the land and ascertain all of the essential facts as to ownership; he should then endeavor to meet the owners, or their representatives, and agree upon a price which may be deemed fair and reasonable. In all such matters great care must be taken to procure favorable terms, and to put them in writing, so as to avoid future misunderstanding.

The vendor is expected to furnish an abstract of title of his property. This abstract must be authenticated in some form. If authority is obtained to carry out a form of procedure now under consideration by the Department, the abstract can be submitted to the United States Attorney for the judicial district in which the land is situated, who will make a thorough examination of the record and the papers, and place thereon his certificate in the form required by the Department of Justice in other like cases. These papers, with his certificate, will then be forwarded to this office by the district engineer.

In case this procedure is not adopted it will be necessary to have the abstract of title certified to by the officers having custody of the

records relating to land titles, judgments, and taxes, unless some other form of giving evidential value to the abstract of title is provided for by the laws of the State or Territory. The district engineer may adopt the latter method if the local laws make it easier.

With the abstract of title there should also be a form of deed which the vendor proposes to execute. In this deed the United States of America should be made the grantee, and it should contain the recital that the purchase is made in pursuance of the provisions of the act of Congress of June 17, 1902 (32 Stat. L., 388). A duplicate of this deed should be retained in the district engineer's office in order that it may be executed if the title is found satisfactory and the form of deed submitted to the Department is approved.

In forwarding these papers to the chief engineer the district engineer will submit a statement covering the following principal points:

First. That the land is necessary for the purpose authorized by the reclamation law.

Second. That the persons with whom a tentative arrangement has been made are competent and authorized to make such tentative proposition.

Third. That the price tentatively agreed upon is fair, and that no more favorable rates can be obtained by ordinary methods, together with a definite recommendation for the purchase. In this connection statements should be given as to the cost or selling price of adjacent or similar property.

Fourth. A statement of the arrangements to be made for the payment of any taxes due or any other lien on the property.

It may be necessary to consider other items in various purchases, and these must be anticipated by the engineer when the papers are transmitted.

These papers are submitted to the Secretary of the Interior with suitable recommendations. They are considered and passed upon by the Assistant Attorney-General for the Department of the Interior. If the title as shown by the papers is satisfactory the chief engineer will so notify the district engineer. The deed should then be executed and recorded, whereupon payment for the land will be made.

Whenever a disbursing officer is in the field at a point convenient to the place where the purchase is made, he may make the payment under the following conditions: The deed, executed in the form approved by the Secretary of the Interior and duly recorded, must be delivered to him, and also certificates from the county recorder and the clerk of the county court showing that no instruments have been placed of record in their offices affecting the title to the lands in question between the date of their certificates forming part of the abstract of title and the date of record of the deed. Upon receipt of these

papers the disbursing officer can make the payment, for which these papers will be sufficient basis, as the letter of the Secretary of the Interior approving the title and the form of deed, and authorizing the purchase, will be attached to his account when it is transmitted to the Treasury Department.

In case there is no disbursing officer available it will be necessary to send the executed and recorded deed to the Washington office of the Survey with the certificates of the county recorder and the clerk of the county court, whence they will be transmitted to the Treasury Department through the Secretary of the Interior. A warrant will then be drawn by the Treasury Department and forwarded to the vendor.

The various operations of transmitting and examining the papers in the several offices require time, and in all cases the party selling the land must be assured that, after the terms of sale have been agreed upon, a reasonable time must elapse for the transmission and examination of the papers and the final payment. This routine will require, as a rule, thirty to sixty days if the title is not defective in the first instance.

Authority to enter upon negotiations for options or for purchase can be procured on application to the chief engineer, and these negotiations are subject to the approval of the Secretary. Various forms of option have been provided for use in the purchase of lands and for acquiring right of way for canals or reservoirs, copies of which can be had upon application. In securing these options it is not customary to make the actual payment of the "dollar" mentioned, and if such payment is made it can not be recovered upon the usual form of account.

Most of the lands purchased are within reservoir sites or at points where the improvements must be ultimately destroyed or abandoned. When it becomes apparent that certain improvements must be abandoned or destroyed, all movable property not needed for the reclamation service should be sold at public auction, in accordance with law and the rules of the Department. Suitable advertisement of the sale should be made, or notices should be posted, and the expense of advertising, services of auctioneer, etc., should be deducted from the proceeds, which should then be transmitted to the chief engineer, to be forwarded, through the proper channels, to the Treasury Department.

LAND AND WATER TITLES.

In each project there are involved tracts of land in private ownership and waters claimed by individuals for purposes of irrigation, power, or other uses, and it is necessary to make a careful examination of the character of the title to the land and of the actual uses of the water. For this purpose suitable blanks and notebooks have been

prepared, and instructions given by various district chiefs. As an example one of these blanks is appended herewith—that used in the State of Washington:

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY.

Report on irrigation canals taking water from ..... River in ..... County, State of Washington.

I ..... do affirm the following to be a true statement of facts concerning the irrigating ditch which I use to irrigate the land described as follows:

.....

.....which I $\frac{\text{own}}{\text{lease}}$ ..... I have ..... acres of land under cultivation, ... acres of which I irrigate.

I have ample supply of water during ..... During the months of ..... I have .....

My canal is ..... miles long and connects with ..... The cost of repairs on my ditch, including my own and all other labor, is ..... per year. I began to use water on the land about ..... and do not need it for purposes of irrigation after ..... I use the water for ..... I generally turn the water off from the ditch .....

I pay $..... annually to ..... for taking water from .....

SIGN HERE .....

In obtaining the above information two principal objects were in view. First, to obtain evidence as to how much water was being used per acre of land irrigated, and, second, to record before any probable legal complications arise a signed statement from the irrigator as to how much land was irrigated and from what sources. In the future there will doubtless arise difficulties in dealing with some of these people and evidence of this kind will be valuable in making settlements.

RIGHT OF WAY FOR CANALS.

The use of public lands for right of way for canals and laterals constructed by the reclamation service is provided for by the act of Oct. 2, 1888 (25 Stat. L., 526). There is also a reservation of right of way for canals constructed by the United States affecting all lands disposed of subsequent to that date. The following circular has been issued concerning the matter:

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
Washington, D. C., August 15, 1903.

To the engineers in charge of projects of the reclamation service:

There is herewith transmitted a circular (9–263) for use in claiming rights of way to which the United States is entitled for ditches or canals constructed by the recla-

mation service. This form has been approved by the Secretary of the Interior, and such copies as may be needed can be furnished on application.

Engineers using this circular will keep a duplicate record of the names of persons to whom they have been sent and the lands specified in each case. One copy of such list should be forwarded to this office and the other retained in the files of the local engineer.

These circulars should be used only after a canal line has been finally located, and it is specially important that they should not be sent out when such action is likely to complicate negotiations for rights of way for other lands. The sending of such circulars will commit the reclamation service to a particular canal location, and if prematurely sent out would encourage those from whom rights of way must be acquired by negotiation to demand greater compensation than they might otherwise ask.

F. H. NEWELL, *Chief Engineer.*

9–263.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
Washington, D. C., ...

Mr.....

.....

.....

SIR:

The reclamation service established under authority of the Secretary of the Interior in pursuance of the act of Congress of June 17, 1902 (32 Stat. L., 388), is about to undertake the construction of a ditch or canal across lands in section..... T. ....., R. ....., ...M., of which it is understood that you are the owner.

An examination of the records of the United States Land Office shows that your entry was made subsequent to October 2, 1888, and that the lands included therein are accordingly subject to the following provision of law:

"That in all patents for lands hereafter taken up under any of the land laws of the United States, or on entries or claims validated by this act west of the one hundredth meridian, it shall be expressed that there is reserved from the lands in said patent described a right of way thereon for ditches or canals constructed by the authority of the United States." (Act of August 30, 1890; 26 Stat. L., 391.)

The entries or claims validated by that act are such as may be asserted under entries made subsequent to October 2, 1888, as held by the Department in the circular of October 5, 1893. (See vol. 17, p. 521, of the Decisions of the Department of the Interior relating to public lands.)

It is accordingly intended to exercise the right conferred by this law, and to take necessary right of way across the said lands for the construction and maintenance of the ditch or canal required in connection with the reclamation work authorized under the act of June 17, 1902.

For information concerning the location of the proposed canal or ditch you are referred to the engineer in charge, Mr....., at.....

You are further informed that you will be entitled to receive water for the irrigation of your lands from the said canal or ditch, in accordance with the provisions of of the said act and regulations authorized thereunder by the Secretary of the Interior.

Very respectfully,

CHAS. D. WALCOTT, *Director.*

NOTICE OF APPROPRIATION OF WATER.

Section 8 of the reclamation act provides that the Secretary of the Interior, in carrying out the provisions of the act, shall proceed in conformity with the laws of any State or Territory relating to the control, appropriation, use, or distribution of water used in irrigation. Before the filing of the first notice of appropriation of water in any State the matter of the advisability of making such filing should be submitted to the chief engineer, because some of the State laws may be such that it is impossible to comply with them in conducting operations under the reclamation act.

The following is a general form of notice of appropriation of water, submitted as a guide for engineers who file such notices. Authority to act in the name of the Secretary of the Interior can be obtained by applying to the chief engineer, who will obtain the same from the Director, the latter being empowered by the Secretary of the Interior to designate suitable persons to act in his behalf.

In many cases it will be necessary to vary this form of notice in order to meet the requirements of the local laws. When such a notice has been prepared it is advisable to submit a copy to the United States district attorney, calling his attention to the instructions of the Attorney-General, dated September 14, 1903, providing for giving advice to the engineers of the reclamation service concerning the preparation of such notices, and requesting any suggestions which he may have to offer in regard to the form submitted. The original notice, with evidence that it has been recorded in the proper State or Territorial office, should be promptly forwarded to the chief engineer.

FORM OF NOTICE OF WATER APPROPRIATION.

....., Engineer, United States Geological Survey, thereunto duly authorized by the Secretary of the Interior, for and on behalf of the United States of America, under the provisions of the act of Congress approved June 17, 1902 (32 Stat. L., 388), claims, at the point where this notice is posted, all the unappropriated waters of the.....River, both surface and underflow, more specifically stated as amounting to.....cubic feet per second.

This notice is posted on....., 190.., on a.....tree on the.....bank of the.....River, in sec..... T..... R....., ...M., at a point distant.....feet and bearing.....from the.....corner of said section.

The water is to be used for irrigation, domestic, power, mechanical, and other beneficial uses in and upon lands situated in.....counties, and located in the following townships: .....

.....

The water hereby appropriated is to be stored by means of a dam located in sec. ....., T....., R....., in a reservoir located in Ts....., R. ....., and will be conducted to the points of intended use by means of canals,

flumes, pipes, tunnels, or other appropriate means of conveying water, of the following dimensions, ..., or such other equivalent dimensions and grade as will give a capacity of.....cubic feet per second.

AFFIDAVIT.

.....being duly sworn, deposes and says that on....., 190.., he did post a notice of appropriation of water, of which the foregoing is a copy, for.....cubic feet per second of the waters of..... River, on a.....tree at a point described as follows: ..... (Describe exactly as in notice.)

Following is a copy of a notice actually filed with the recorder of Riverside County, Cal., and posted as indicated:

Notice is hereby given that Edmund T. Perkins, thereunto duly authorized by the Secretary of the Interior, for and on behalf of the United States of America, and under the provisions of an act of Congress approved June 17, 1902, claims at the point where this notice is posted all unappropriated waters of the Colorado River, both surface and under flow, to the extent of 100,000 cubic second-feet—4,000,000 miners' inches. The point where this notice is posted is described as follows: A point of rocks known as Black Point, in California, county of Riverside, about section 7, township 5 south, range 24 east, San Bernardino meridian, California. The notice is posted on a box nailed to a tree.

The said water is claimed for irrigation, manufacturing purposes, water power, domestic use, and other beneficial uses, and the terminus and place of its intended use is in the valley extending from said point along the Colorado River to and below Picacho, Cal., and along the line of a proposed conduit. The water will be diverted by means of a diversion dam and will be conveyed through canals, flumes, wooden, iron, steel, and cement pipes to the place of intended use. The size of the canal will be 100 feet wide on the bottom and 10 feet deep, or of such dimensions as will give it an equivalent capacity. The size of the conduits are to be such as to give them sufficient capacity to carry the amount of water required.

LAND CLASSIFICATION.

The detailed topographic survey of irrigable lands should be followed at once by a classification of such character that it will be possible to divide the lands into farms capable of supporting a family. A farm unit may be 40, 80, 120, or 160 acres. Lots as designated on the official township plots may be used by themselves or with 40-acre tracts. Sufficient knowledge must be had of each 40-acre tract to enable the preparation of a map of each farm.

The subdivision of the irrigable lands into farm units is one of the most important duties of the reclamation service. The following circular of September 3, 1903, has been issued as a guide for such work:

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
Washington, D. C., September 3, 1903.

To the Engineers in Charge of Reclamation Projects:

By decision of the Secretary of the Interior, under date of August 21, the reclamation service is authorized to classify and combine the different subdivisions within lands withdrawn for irrigation under the reclamation act so as to make practically

homestead segregations of the entire territory under any one project, specifying the particular legal subdivisions that shall constitute each entry as to all public land included in the withdrawal. Such segregations of homestead entries must be by legal subdivisions (40-acre tracts or lots) and be limited to areas of not less than 40 nor more than 160 acres. All the subdivisions forming each entry must be of contiguous tracts. Two tracts containing only a corner in common are not contiguous, nor in general are they contiguous when separated by an existing meandered stream. The homestead entries so segregated may be of different areas in the same project, the intent being to provide in each homestead entry such quantity and quality of irrigable land as shall be sufficient for the support of a family.

This classification of the lands will be reported by the Director of the Geological Survey to the Secretary of the Interior for incorporation in the public notice of the lands irrigable under each project, as required by section 4 of the reclamation act. In submitting such list the area of irrigable land in each entry should be specified, as it will form the basis of the payment required for the water supplied from the irrigation works of the reclamation service.

Very respectfully,

F. H. NEWELL,
Chief Engineer

In order that the lands withdrawn for the purposes of the reclamation service may be protected against acquisition by parties making desert-land entries before the withdrawal became effective, the Secretary of the Interior has ordered that before the acceptance of final proof on desert entries within withdrawn areas, the land shall be examined and reported on by engineers of the reclamation service.

The restoration and re-marking of public-land-survey corners by the engineers and topographers of the reclamation service on lands withdrawn under the reclamation act has also been authorized.

The circular of July 21, 1903, covering both of these subjects, follows:

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY, RECLAMATION SERVICE,
Washington, D. C., July 21, 1903.

FINAL PROOF OF DESERT-LAND ENTRIES.

By order of the Secretary of July 6, 1903, the reclamation service will report on all desert-land entries for which final proof is offered when the lands involved are included within withdrawals under the reclamation act. This office will be promptly advised of the offer to make such final proof; you will be notified thereof and will, as soon as possible thereafter, have the land inspected by one of your assistants upon whose discretion and judgment you can rely. It is important that your report be promptly forwarded to this office, in order that there shall be no delay in the disposition of these cases by the General Land Office.

Inclosed herewith are several copies of a circular containing all the provisions of law concerning desert-land proofs, together with the regulations thereunder. Other copies can be furnished if desired.

PUBLIC-LAND SURVEYS.

By order of the Department of July 14, 1903, the engineers and topographers of the reclamation service are authorized to restore and re-mark any corners of the public-land surveys which may be found to have been destroyed or obliterated in

whole or in part upon the lands included within withdrawals under the reclamation act. In the work done in this connection the rules in the circular on the Restoration of Lost or Obliterated Corners, issued by the General Land Office, March 14, 1901, and the requirements of the Manual of Surveying Instructions for the Survey of Public Lands issued by the General Land Office must be followed.

In restoring corners it is important that there shall be no interference with vested rights where the lands have been settled upon and that no corner should be regarded as missing until diligent search has been made for evidences of its original location.

For this work it will generally be necessary to have copies of the field notes on file in the office of the surveyor-general of the State or Territory in which the work is located.

The topographers and engineers should keep careful and complete record of field notes of the work with descriptions of corners established, in the manner required by the regulations of the Land Office. Such field notes should be kept on file in the office of the engineer in charge.

No corners are to be reestablished or re-marked unless they are required for the classification of irrigable lands or for the identification of lands the title to which must be acquired.

There is herewith transmitted a copy of the circular on the Restoration of Lost or Obliterated Corners, approved March 14, 1901, and also a copy of the Manual of Surveying Instructions. Additional copies of these circulars can be obtained on request, but you are advised that copies of the manual should be carefully preserved, as the published edition is not very large.

F. H. NEWELL, *Chief Engineer.*

General suggestions as to the withdrawal and classification of lands have been given in the book of Instructions, beginning on page 96. Additional suggestions are desired from men in the field, such as may lead to the preparation of more specific instructions, based upon experience.

Cooperation is being had in some cases with the Bureau of Soils of the Department of Agriculture, and memoranda have been prepared with reference to the examinations of lands along Colorado River:

FORMATION OF ASSOCIATIONS.

When it appears probable that any particular project is practicable and will be carried into effect, steps should be taken looking toward the organization of an association of the people owning land under the project. This law is very general in terms, and it is necessary to supply by administrative action a body of rules and regulations covering such associations.

The object of these associations is to enable prompt and effective dealing between the Secretary of the Interior, represented by the engineers of the reclamation service, and the water users. The latter should have a common agent with whom business may be transacted, so that it will be practicable to reach definite conclusion on points that may be at issue. It is impossible to deal satisfactorily with the indi-

viduals of a community where the members have had little or no experience in irrigation, and where there is wide diversity of opinion on matters of detail. When an association is formed, in which the majority rules, it is possible to refer to it all questions relating to water distribution and all other questions that form sources of individual discontent, leaving these to be settled under State laws or customs, or according to the wishes of the majority in interest.

Much time and thought has been devoted by many persons to the formation of these associations, and in particular the matter has been worked out with great care in the Salt River Valley, Arizona. The results of this work will be presented by Mr. B. A. Fowler, the president of the Salt River Valley Water Users' Association, and reference should be made to pages 130–138 for the principal facts.

WORK IN HAND.

The following paragraphs give briefly the present condition of the work in each of the sixteen States and Territorities. This is being pushed forward as rapidly as possible consistent with economy of administration and efficient organization of the experienced men available.

In Arizona detailed surveys in connection with the proposed dam on Salt River, at the mouth of Tonto Creek, have been continued, and plans have been made for a power plant, cement mill, and other structures essential to the construction of the great dam. An organization of the water users has been effected, and ample time is afforded for holders of arid land to indicate their wishes as regards obtaining a supply of water from the Government reservoir, if constructed. The public lands in the Salt River Valley have been withdrawn from entry under the terms of the law, to be reclaimed if there should be sufficient water for any considerable part of these. An alternative project—that near San Carlos, on Gila River—has also been under consideration, and facts have been obtained concerning its feasibility. Other surveys and reconnaissances have been made in various localities, notably along Colorado River.

In California the most important work done has been that in connection with the storage and utilization of the waters of Colorado River, in the southeastern part of the State. Any project here must probably be of considerable magnitude, the complete utilization of the river being in some respects analogous to the achievements of the British engineers on the Nile, in Egypt. Examinations have also been made in the Sacramento and San Joaquin valleys, and facts have been obtained upon which to base a comparison of various projects and a determination of their relative cost and importance.

In Colorado attention has been given principally to the tunnel which it is proposed to build to convey the waters of Gunnison River to the Uncompahgre Valley, in the vicinity of the towns of Montrose and Delta. An examination has also been made of the proposed high-line canal from Grand River, near Grand Junction, and of the feasibility of diverting and storing the flood water of South Platte River in the vicinity of Sterling. A general reconnaissance has also been begun in the northwestern part of the State.

In Idaho opportunities have been found at several places for reclaiming arid land by using the waters of Snake River and its tributaries, especially by utilizing as reservoirs the lakes at the headwaters of Snake River. A large tract of land has been reserved on the Snake River Desert, in the vicinity of the town of Dubois; another tract reserved lies below American Falls, near Minidoka; a third project contemplates the irrigation of lands lying between Boise and Payette rivers, west of the city of Boise.

In Kansas a number of small projects have been examined, but none of these appear to fall within the scope of the reclamation law. A very thorough study has been made of western Kansas and of the Great Plains in general, in order to ascertain the opportunities for obtaining deep or artesian water. A map showing the depth to the water-bearing strata and the probability that the water will rise to the surface has been prepared and is in process of publication.

In Montana surveys have been made for canals along Milk River, and steps have been taken to procure the necessary reservoir sites. A general reconnaissance has also been begun in the central part of the State, between Missouri and Yellowstone rivers, and examination has been made of the opportunities for diverting the Yellowstone in the eastern part of the State.

In Nebraska the principal opportunities for reclamation are bound up with the complete use, storage, and utilization of the waters of North Platte River. The reservoir sites are in the State of Wyoming. An examination has been made of these, and also of the feasibility of covering arid land in the western part of Nebraska by canals heading in Wyoming.

In Nevada construction has been begun, as already stated, in the Truckee Canal, taking water from the river of this name, the head of the canal being in the canyon below Reno and above the town of Wadsworth. This canal will take the flood or excess waters of Truckee River to a reservoir site on the lower part of Carson River, from which the water can be conducted out upon the desert lands lying to the east.

In New Mexico a number of surveys have been made upon Pecos River and the Rio Grande. On the former stream the principal locali-

ties are above Roswell, at what is known as Urton Lake, and below Roswell on Hondo River. Maps of reservoir sites on the Rio Grande above the Mesilla Valley are also being prepared.

North Dakota presents few problems of national importance. The principal water supply of the State is found in Missouri River, the fall of which is so gentle that canals can not be taken from the stream. The tributaries of the Missouri in this State are small or intermittent, or are bordered by valleys so narrow that any considerable development of irrigation does not seem practicable. Pumping may be resorted to, and for this purpose lignite may be obtained. The efficiency of this method of irrigation is being investigated.

Oklahoma is in many respects similar to North Dakota, in that large reclamation projects are impracticable or unnecessary. The greater part of the Territory lies east of the one-hundredth meridian and irrigation is not regarded by settlers as playing an important part in the agricultural development of its area. Beaver County, extending from the one-hundredth to the one hundred and third meridian, is to a large extent arid, but water must be supplied to these lands by the diversion and storage of flood waters, which is of doubtful practicability on a large scale.

In Oregon important detailed surveys are being made in three localities, one of these being in Harney County, near the central part of the State; another on Malheur River, on the east; and the third on Umatilla River, near the northern boundary. Each of these projects has certain advantages and difficulties, and a very careful examination must be made before the feasibility and relative merits of these can be known.

In South Dakota there are apparent opportunities for reclamation in areas near the Black Hills. Several small streams issue from this group of hills, and storage of flood waters is believed to be practicable. The principal opportunity for reclamation thus far discovered is that near Belle Fourche, but other irrigable lands, mainly in private ownership, have been noted east of Rapid City.

In Utah the people, through a State commission, have asked that first attention be given to Utah Lake. This body of water receives the drainage from a considerable part of the mountain area in the center of the State, and the greater portion of this water is lost by evaporation. The problem is largely to prevent by one means or another this enormous waste, which has been estimated to be equivalent to 1,000 cubic feet per second during the irrigating season. Another project under examination contemplates the use of Bear Lake, which lies partly in Idaho and partly in Utah.

In Washington the great body of irrigable land is in the central part of the State, within what is known as the great bend of the

Columbia. This land is below the level of Spokane River, at the foot of Spokane Falls, and may be reached by water from this or other sources if the cost is not prohibitory. The scheme is gigantic in its proportions, and its feasibility can not definitely be determined without careful surveys. Similar bodies of land north of the Columbia and near Okanogan can be reclaimed probably at less cost.

Wyoming contains mountain masses from which come several of the larger rivers of the arid region. The altitude is high, but there are a number of opportunities for reclaiming desert land. The most notable of these is in the vicinity of Cody, in the Bighorn Basin. Surveys have been made in this area, and also on the east side of the Bighorn Mountains, to ascertain the possibility of storing water in Lake De Smet. On North Platte River also extensive surveys are being made to determine the possibility of storing its flood waters for use in eastern Wyoming and western Nebraska.

As each of the projects above mentioned contemplates large works, requiring construction that is more or less involved in character, and as hydraulic works are difficult to build, and all questions of safety and stability must be settled in advance with great care, it is necessary to make examinations with unusual thoroughness. Instructions are given to ascertain all of the facts and to have every essential feature guarded in the most thorough manner possible. In this work it is better to run the risk of criticism for slow progress than rush into expensive yet inadequately considered construction, especially where safety is involved.

PRINTED FORMS.

Some of the forms for customary use in the various operations are here presented for general information and discussion.

FORM OF CONTRACT.

THIS AGREEMENT, made and entered into this ..... day of ...., one thousand nine hundred and ....., by and between the United States of America, acting in this behalf by....., Secretary of the Interior, party of the first part, and ....., party of the second part, his heirs, executors, administrators, and assigns,

WITNESSETH, That in pursuance of authority for the construction of irrigation works contained in the act of Congress approved June 17, 1902 (32 Stat., 388), entitled "An act appropriating the receipts from the sale and disposal of public lands in certain States and Territories to the construction of irrigation works for the reclamation of arid land," the parties hereto have mutually covenanted and agreed, and by these presents do covenant and agree, to and with each other, as follows:

ARTICLE 1. The second party agrees, under penalty of a bond conditioned upon a due compliance with the provisions of this agreement, to construct and complete .....

.....

.....

in accordance with terms of the attached advertisement, proposal, and specifications, the same being made a part of this contract, excepting in so far as the specifications may be amended or modified by the terms of this agreement.

ARTICLE 2. For and in consideration of the faithful performance of the stipulations of this contract, the second party shall be paid, upon the presentation of proper accounts approved by the Chief Engineer of the Reclamation Service, the Director of the Geological Survey, and by the Secretary of the Interior, in the following manner:

ARTICLE 3.—The contractors must save, keep, bear harmless, and fully indemnify the United States and any of its officers or agents from all damages or claims for damages, costs, or expenses in law or equity that may at any time arise or be set up for any infringement of the patent rights of any person or persons in consequence of the use by the Department of the Interior, or by any of its officers or agents, of articles supplied under this proposal, and of which the contractor is not the patentee or assignee, or lawfully entitled to sell the same.

[Here follows space for inserting additional articles.]

ARTICLE—. It is expressly understood by the second party that, in conformity with the requirements of section 3737 of the Revised Statutes, neither this contract nor any interest therein shall be transferred to any other party or parties, and that any such transfer shall cause the annulment of the contract so far as the United States is concerned; all rights of action, however, for any breach of this contract by the contracting parties being reserved to the United States.

ARTICLE—. It is further expressly understood that no Member of or Delegate to Congress, officer, agent, or employee of the Government is or shall be admitted to any share or part of this contract, or to any benefits which may arise therefrom, and the provisions of sections 3739, 3740, and 3741 of the Revised Statutes of the United States, relating to contracts, enter into and form a part of this agreement so far as the same may be applicable.

In witness whereof the undersigned have hereunto subscribed their names and affixed their seals the day and year first above written.

[DEPARTMENT SEAL.]

....., [SEAL.]
Secretary of the Interior, for and on behalf of the
United States of America, party of the first part.

Two witnesses to each signature.

.....
P. O. address.....

.....
P. O. address.....

(Executed in quintuplicate.)

....., [SEAL.]
Party of the second part.

The preamble will have the following form in the case of a firm:

THIS AGREEMENT, made and entered into this ..... day of ....., one thousand nine hundred and ....., by and between the United States of America, acting in this behalf by ....., Secretary of the Interior, party of the first part, and ....., and ....., partners, doing business under the firm name and style of .....,of ....., ..., party of the second part, their heirs, executors, administrators, and assigns.

The preamble will have the following form in the case of a corporation:

THIS AGREEMENT, made and entered into this ..... day of ....., one thousand nine hundred and ....., acting in this behalf by .....

....., Secretary of the Interior, party of the first part, and.....
....., of ....., ...
....., a corporation duly organized and existing in the State of.....
....., party of the second part, its successors and assigns.

FORM OF BOND

The following is a form of bond used when the contractor (principal) is an individual. A similar form is used, with appropriate changes, when the principal is a firm or a corporation.

BOND.

Know all men by these presents, that we, ...
.....
of ....., county of ....., and
State of ....., principal, and .....
of ....., county of....., and
State of ....., and .....
of ....., county of ....., and
State of ....., sureties, are held and firmly bound unto the United States of America in the sum of .....
dollars ($.....), lawful money of the United States, for which payment, well and truly to be made, we bind ourselves and each of us, our, and each of our, heirs, executors, administrators, and assigns, for and in the whole, jointly and severally, firmly by these presents.

Sealed with our seals and attested by our signatures, at .....
this ..... day of ....., in the year of our Lord one thousand ..... hundred and .....

The nature of this obligation is such that if the said .....
.....
his heirs, executors, administrators, and assigns, or any of them, shall, and do in all things well and truly observe, perform, fulfill, accomplish, and keep all and singular the covenants, conditions, and agreements whatsoever, which on the part of the said .....
his heirs, executors, administrators, and assigns, are, or ought to be, observed, performed, fulfilled, accomplished, and kept, comprised, or mentioned in certain articles of agreement, bearing date the ..... day of ....., one thousand ..... hundred and ....., between the said .....
.....
and the said ....., Secretary of the Interior, concerning the .....
.....
.....
.....
.....
according to the true intent and meaning of said articles of agreement, and shall promptly make payments to all persons supplying labor and materials in the prosecution of the work provided for, then the above obligation to be void; otherwise, to remain in full force and virtue.

In testimony whereof, the said.....
as principal.. and.....
.....
....., as sureties,

have hereunto subscribed their hands and affixed their seals the day and year first above written.

Signed, sealed, and delivered in the presence of—

[Two witnesses to signature of principal and each surety.]

.....
(P. O. address): .....
.....
(P. O. address): .....
.....
(P. O. address): .....
.....
(P. O. address): .....
.....
(P. O. address): .....

..... [SEAL.]

..... [SEAL.]

..... [SEAL.]

[Principal and sureties sign and affix seals of wax or wafer.]

STATE OF.....
COUNTY OF..... } ss.

Personally appeared before me a..... .....
in and for said..... the said .....
.....
[Here insert names of principal and sureties.]
.....
who signed the foregoing obligation, and who made solemn oath that they and each of them own and possess property not exempt by law from execution, over and above their debts and liabilities and free from incumbrances of the value of the sum of ..... dollars.

[Principal and sureties sign.]
.....
.....
.....

Sworn to and subscribed as above written before me this ..... day of ....., 1.....

[L. S.] [Magistrate's signature.][a].....
.....

☞If the bond is executed in the District of Columbia, the following certificate must be signed by a United States officer *other than a notary public.* If executed elsewhere, by a judge of the United States court, or a United States district attorney of the district in which the sureties reside.

.....District.....
....., 1.....

[a] If executed outside District of Columbia, authority of the officer administering oath must be certified by the clerk of a court of records under seal of court.

I hereby certify that.....

.....

the sureties who have signed the foregoing bond are known to me as residents of .....,, and citizens of the United States, and that I believe them to be amply sufficient security for the amount thereof, and that the bond is good.

REGULATIONS CONCERNING CONTRACTS AND BONDS.

The following paragraphs contain the regulations governing the preparation and execution of contracts and bonds. These are taken from departmental form No. 1–366, dated April 8, 1903, and signed by Hon. E. A. Hitchcock, Secretary of the Interior:

CONTRACTS.

1. Every contract must be reduced to writing and signed by the contracting parties at the end thereof.

2. The post-office address of each party and witness must appear in the contract, all dates plainly written and blanks carefully filled, and all particulars and conditions stated as fully and clearly as practicable.

3. Every contract must contain the following clause: No Member of or Delegate to Congress, or any other officer or agent of the United States, either directly or indirectly, himself, or by any other person in trust for him, or for his use and benefit, or on his account, is a party to or in any manner interested, in whole or in part, in this contract, or in the enjoyments, benefits, profits, or emoluments arising therefrom. (Rev. Stats., secs. 3739–3742, inclusive.)

4. Erasures, interlineations, or other irregularities must be explained over the signature and seal of the parties to the instrument, and witnessed by the officer before whom the same is executed.

5. No assignment or transfer of any contract, or interest therein, will be recognized by the Department, and such assignment or transfer will operate as an annulment of the contract so far as the United States are concerned or interested therein.

6. All contracts must be executed in duplicate, unless otherwise directed.

7. Whenever the contract is between the Government and a copartnership, in addition to the firm name, each individual member thereof must sign his own name thereto; the firm name alone will not be sufficient, and wherever the firm name appears in the contract it should be indicated thus: A, B, C, and D, partners doing business under the firm name of (here insert firm name), and the contract must be signed in the same manner.

8. Where the contract is between the Government and a corporation or an incorporated company, it may be executed in the name thereof by the officer authorized so to do by law or resolution, but in all such cases evidence shall be furnished the Department of the corporate existence of such company or incorporation, together with a certificate from the secretary or other officer having charge of the seal thereof, that the board of directors or executive committee duly authorized the making of the contract for and on behalf of the company by such individual; the act of such officer in executing the contract shall be binding upon the company or corporation represented by him, and the manner of fixing the signature thereto must be by first signing the name of the company or corporation, followed by the signature of the party authorized to act therefor, thus: (Name of company) ——— ———, by ——— ——— (giving official character).

9. In the execution of contracts the names of all the parties thereto must be written in full and an impression or seal affixed to each; signatures by initials will not

be accepted; each of the signatures of the contracting parties must be witnessed by at least *two* persons, and the signatures of the witnesses should be written opposite the signatures witnessed.

10. No contract signed by an agent or firm, or by one member thereof for another, or by an attorney, will be accepted, unless the execution thereof be accompanied by a duly executed power of attorney from the party or parties for whom such agent, member, or attorney shall act, which must specifically set forth the purpose for which it was given, describing in full the names of the contracting parties, the date, and object thereof.

BONDS.

1. Each contract must be accompanied by a joint and several bond in the sum of at least 50 per cent of the amount of the contract price, executed by the principal and at least two individual sureties conditioned for the faithful performance of the terms thereof; but a corporate surety duly qualified under the act of Congress of August 13, 1894, may be accepted as sole surety. Corporate sureties on bonds are preferred.

2. The full name of the principal and each of the sureties should be written in the body of and signed to the bond, and there must be a seal of wax, wafer, or other adhesive substance attached to the signature of the obligor and each of the sureties. The printed words "seal" or "scroll" will not be deemed sufficient.

3. The principal and each of the sureties must sign the bond in the presence of two persons, who shall sign their names thereto as attesting witnesses, and it must appear for whom each witness signs, but the same witnesses may witness all the signatures, if signed in their presence.

4. The sureties must justify in amounts the aggregate of which will be equal to at least twice the penalty of the bond. This rule applies to corporate as well as to individual sureties; and corporate sureties will be required to attach to each bond a copy of the last statement of their assets and liabilities as rendered pursuant to section 4 of the act of Congress of August 13, 1894, together with a certificate, under corporate seal, by the secretary of the corporation, as to the official character of the person signing for the corporation, and that such person is so authorized.

5. Where a party or parties become surety or sureties to more than one bond, evidence must be filed to satisfy the Department that the said party or parties are possessed of property not exempt from execution over and above all debts and liabilities, and free from all incumbrances of value not less than the sum of the several bonds for which they are sureties.

6. Whenever such affidavit is made before a notary public or justice of the peace outside of the District of Columbia, the jurat shall be accompanied by a certificate of the clerk of the district or circuit court under the seal thereof, in the county and State in which such notary or justice resides, that at the time of administering such oath such notary or justice held such office and was duly authorized by law to administer such oath.

7. No person who is a bonded officer or married woman will be accepted as a surety, but unmarried women may be accepted as such, provided that the officer administering the oath certifies that they possess sufficient property in their own right to make them sufficient sureties, and that they are single.

8. Where a corporation is a party to a contract, no officer thereof will be accepted as surety thereon, and the same rule applies where a copartnership is a party to a contract.

9. Where the sureties are other than corporate, it must appear in the affidavit or oath of justification that such surety is more than 21 years old and in all other respects legally competent to make and enter into a contract.

10. Bonds may be executed before a clerk of a court of record, or before any other officer authorized to administer oaths, by observing rule 6 herein.

11. All bonds must be accompanied by a certificate as to the sufficiency of the sureties thereon, signed by a judge of a United States court, a United States district attorney, a United States postmaster (or such other officer of the United States as may be acceptable to the Secretary of the Interior), residing in the district where such bonds are executed. The foregoing does not apply to corporate sureties who have complied with the requirements of paragraph 4.

12. Special pains must be taken to prevent erasures or mutilations of any kind in any bond, but where made they must be explained by a certificate of the officer before whom the instrument is executed "that such erasures or mutilations were made before the bond was signed by the obligor and the sureties."

13. Any contract or bond not executed in conformity with the foregoing regulations will be returned for an observance thereof.

[At the conclusion of the address by the chief engineer there was a general discussion which was participated in by the engineers present, numerous questions being asked, the answers to which have been now embodied in the text of the paper presented in the preceding pages. A general programme for discussion was then outlined, the States and Territories being taken up in alphabetic order.

As a preliminary to this systematic discussion of work in the field short addresses were made by Mr. N. H. Darton, geologist, upon the underground waters in the reclamation States; by Mr. Gifford Pinchot on the relation of forestry to irrigation, and by others present on various phases of the work.]

PAPERS READ AT THE CONFERENCE.

WESTERN HYDROLOGY.

By N. H. Darton.

[The western section of the division of hydrology was created in the early part of 1903 to make a systematic investigation of underground water resources of all kinds and to collect all data relating to geology that have been developed by deep borings. There was and is a very large demand upon the Survey for information regarding prospects for using underground waters, and as the reports available covered only a few restricted areas it was evident that there was great need for an extended study of this important resource. As the investigation was almost entirely a geologic one, it was placed in the hands of geologists. Fourteen parties have been at work in the field, the area under investigation comprising the arid lands of the reclamation States and Texas.

Much of the work of the western section of hydrology is closely related to the reclamation service, as a large share of attention will be given to areas in which underground waters may be brought to the surface and employed in irrigating lands that are now unproductive, many of which are so situated that they can not be economically irrigated by the diversion or storage of surface waters. Some of the operations of the section and some of the results accomplished are here given, by States.]

Arizona.—Mr. Willis T. Lee has made a very detailed examination of the geology and underground water resources of a portion of Salt River Valley, mainly lying south of Phoenix, and the preparation of a report on the results is now far advanced. The wells have been investigated and the light which their records throw on the underground geology has been duly considered. The volume of water available, the prospects for obtaining additional supplies from deeper horizons, and the conditions under which these waters may be pumped to the surface and used for irrigation have received due attention. Mr. Lee has also explored the Hualpai, Chino, Sacramento, and Aubrey valleys in western Arizona with a view to ascertaining the prospects for obtaining water supplies from the detrital materials with which these valleys are filled. Further work is to be done in Arizona during the winter to

determine the geologic structure and underground water conditions in various portions of the Territory.

California.—Mr. W. C. Mendenhall and assistant are engaged in a detailed investigation of the geology and underground water resources of the region about San Bernardino and Riverside. It is expected that the results of these studies will throw light upon the conditions under which waters occur in the detritus-filled valleys and the broad, flat, alluvial fans at the foot of the mountain slopes in southern California.

Colorado.—Considerable attention has been given to obtaining additional information regarding the prospects for artesian flows in a portion of Colorado lying east of the Rocky Mountain front, and numerous measurements have been made of the strata on their upturned edges along the Front range and Twin Butte uplift. Considerable new information was obtained as to the extent of districts in which flowing waters are obtainable, and one area was found in Prowers County which was not heretofore regarded as within the artesian belt.

Early in September Mr. C. E. Siebenthal began work in the San Luis Valley artesian area. He is making a canvass of artesian wells and determining the geologic structure. The purpose of his investigation is to determine the source, extent, geologic conditions, and volume of the artesian waters.

Kansas.—Some deep borings are being sunk in the northwestern portion of Kansas by persons hoping to find oil, and as these wells will afford much evidence on the underground geology and prospects for water they are being watched very closely as to materials penetrated and water horizons developed.

Nebraska.—During the summer Prof. G. E. Condra, of the State University, has examined the geology and wells in the northeastern portion of Nebraska. He has determined the extent of the artesian basin, and his results will throw much needed light on the structure and water prospects of a wide area southward.

Prof. E. H. Barbour, State geologist, has continued his compilation of information relating to underground water resources of the entire State, and will soon submit a report giving information of all important wells.

New Mexico.—Prof. C. R. Keyes has prepared a report on the geology and underground water supplies of portions of the Rio Grande Valley, in the southern part of New Mexico, which will soon be ready for publication. A part of the expense of this work is paid out of Territorial funds.

Prof. C. N. Gould made an expedition in September into the northeastern portion of the Territory to study the relations of the "Red Beds," which he followed to the Rocky Mountain front. One of the most important problems in eastern New Mexico is to ascertain the prospects for water in these "Red Beds" and underlying formations, and, as

there is great need for water in the district, special consideration is to be given to this question.

North Dakota.—Prof. D. E. Willard has continued his studies of the geology and underground waters in the Red River Valley region in North Dakota. He has obtained additional data to complete the report and folios of the late C. M. Hall, and has extended the investigation into adjoining areas.

Prof. F. A. Wilder, former State geologist, has examined the lignite deposits in the western portion of the State, to ascertain to what extent they may prove available as fuel in pumping water for irrigation. The results of his examination will soon be ready for publication.

Oklahoma.—Prof. C. N. Gould has continued his studies of the "Red Beds" of Oklahoma and their water supplies. In August and September he made an expedition to examine the western portion of the Territory and adjacent regions.

Oregon.—Prof. I. C. Russell has made an expedition through central Oregon to determine the geologic structure and ascertain the prospects for underground waters. Artesian conditions were discovered at several localities where their development would be of considerable importance.

South Dakota.—Prof. J. E. Todd, State geologist, has continued his canvass of South Dakota for well data through the eastern portion of the State, and has obtained a large amount of important information bearing on the extent of the artesian area and the underground geology. He is giving much attention to the collection of evidence on diminution of flow and pressure in some districts to ascertain its cause.

Texas.—Mr. G. B. Richardson has continued his work in Texas, jointly with the State mineral survey, to ascertain the geology and prospects for water in deep wells in El Paso County and a portion of Reeves County. He has made excellent progress, and has obtained a large number of data on the rocks and their structure.

Washington.—Mr. Henry Landes, State geologist of Washington, has made a canvass for all available data as to underground waters in the State, and will soon have a report ready for publication.

Wyoming.—The principle work in Wyoming has been done by Mr. C. A. Fisher, in the Bighorn Basin. The geologic relations and structure have been studied with care, and several horizons of underground waters have been found to exist. Much additional work, however, will be required before the investigation can be completed.

Further observations have been made along the Laramie Front range to determine the thickness of the strata that dip under the plains.

In the region north of the Black Hills considerable work has been done, mainly by Prof. C. C. O'Harra, and it has been discovered that

artesian waters may be expected at moderate depths in an area of considerable size in the valley of Little Missouri River and some of its branches near the Wyoming-Montana State line.

FORESTRY AND IRRIGATION.

By GIFFORD PINCHOT, Chief Forester.

[Mr. Pinchot spoke of the importance of conference between the members of the reclamation service and those of the Bureau of Forestry in order to bring about the most economical and effective administration of the forests and arid lands. In brief, he made the following statement:]

In the delicate and important task of administering the forest reserves, which are so vitally necessary to the well-being of industries and communities throughout the West, the Government is constantly met by the demands of conflicting interests. In each case it is essential that the administration of the reserves should meet the needs of the people; that each reserve should be maintained in its highest possible present usefulness, yet protected with the greatest care in its continued utility to the community.

The idea of permanent usefulness is fundamental in every case. The four great timber-using industries—lumbering, building, mining, and transportation—are peculiarly dependent for their future upon a sustained supply of timber from the Government forest reserves, and special provisions must be made to meet their needs. The use of the mature timber is often required, and is almost universally advisable in a healthy, growing forest, and provision is made by the law for its disposition to those who need it. The use of the reserves will grow steadily greater and more necessary as time goes on, and every care should be taken to safeguard the young growth for the future.

For the present much the most important use of the forest reserves is to supply water to the irrigator, and their utility in this respect should be preserved in every possible way. This use, too, will increase with time, and it will become more and more evident that the foundation of the irrigation development of the West lies in the wise administration of the forest reserves. Not only can the present supplies of water be conserved by the right handling of the forest, but there is no question whatever that in many localities they may be largely increased. From this point of view as well as from many others, protection from fire is the first duty of the Government toward these reserves.

[Mr. Pinchot also called attention to the message of President Roosevelt to the Irrigation Congress, and spoke of the interest shown by the President in the work of the foresters and engineers. The following telegram from the President was received by the congress:]

TELEGRAM FROM THE PRESIDENT.

OYSTER BAY, N. Y., *September 15, 1903.*

To the Irrigation Congress, Ogden, Utah:

The passage of the national irrigation law was one of the great steps not only in the forward progress of the United States but of all mankind. It was the beginning of an achievement so great that we hesitate to predict the outcome. But it was only the beginning. Now that the law is an accomplished fact it must be given effect. To that end the reclamation service, organized under the national irrigation law of June 17, 1902, has been pushing its surveys and examinations of possible irrigation projects energetically in each of the thirteen States and three Territories named in the act. Some of the projects which promised well at first are found on careful study to be impracticable, either because of scanty water supply or of great cost; others must await higher values in land; while still others stand the test and are ready for immediate construction. The feasible projects are always large and costly, because private enterprise has already seized upon the smaller and less expensive ones, leaving to the Government the great works which are to bear so essential a part in bringing the nation to its full development. Great care and highest engineering skill are required to plan and build such works, which are among the most difficult undertakings of mankind. They must be built for permanence and safety, for they are to last and spread prosperity for centuries. To design and build such works a body of engineers of the highest character has been brought together in the reclamation service, for only men impartially selected for capacity alone are capable of creating these great structures.

Merit must govern not only in the selection of the men, but still more in the selection of the projects. Every reclamation project selected for construction must possess the qualities which commend it as a national undertaking, certain to reclaim large tracts of arid land and to support in well-being a dense and vigorous population.

Vast though the benefits of the reclamation law will be, many disappointments necessarily await both the advocates of special projects and the men whose desire for accomplished results outruns the slow and steady development of these great undertakings. It should be borne in mind that a broad survey of all possible projects gives new conception of their relative value, and that a work of prime importance to one group of men may seem less desirable in the light of wider knowledge. Nor is it wise in large affairs to begin construction first and to elaborate details afterwards. Each important point must be carefully studied in advance and the whole plan tested and approved before work can begin. Yet if we proceed both cautiously and persistently under this beneficent law we may confidently expect the largest possible development of our arid lands and their settlement by industrious, prosperous, self-respecting men and women, who will exchange the products of irrigated agriculture for the products of mills and factories throughout the United States. Such communities, flourishing in what is now the desert, will finally take their places among the strongest pillars of our Commonwealth.

The irrigation development of the arid West can not stand alone. Forestry is the companion and support of irrigation. Without forestry irrigation must fail. Permanent irrigation development and forest destruction can not exist together. Never forget that the forest-reserve policy of the National Government means the use of all the resources of the forest reserves. There is little profit in destruction compared with use. The settlement of the great arid West by the makers of homes is the central object both of the irrigation and the forest policies of the United States. In forestry, as in irrigation, the immediate private interest of some individuals must occasionally yield to their permanent advantage, which is the public good. The benefits of forestry are not only for the future but for the present. The forest reserves are for all the people, but first for the people in their immediate neigh-

borhood, for whom supplies of wood and water are among the prime necessities of life. The wiser and more skillful the management of the reserves by trained men, the greater obviously will their usefulness be to the people. We must never allow our chagrin at temporary defects and difficulties in the management of the forest reserves to blind us to the absolute necessity of these reserves to the people of the West. Support of the forest-reserve policy has grown with wonderful rapidity in the West during the last few years. It will continue to grow till the last vestige of opposition, now almost gone, has wholly disappeared before a true understanding of the object and effect of forest reservation. The greater the support of the forest reserves by the people of the West, the greater the assurance that the national irrigation policy will not fail; for the preservation of the forests is vital to the success of this policy.

THEODORE ROOSEVELT.

COLONIZATION.

By FREDERICK DE L. BOOTH TUCKER.

[Comdr. Frederick de L. Booth Tucker, of the Salvation Army, addressed the engineers on the subject of colonization, showing the great importance to the reclamation work of systematic and careful colonization of the vacant land. After the works are completed the great problem will be to organize and control the diverse elements—to consolidate the settlers into a community competent to utilize the great works. His remarks, in brief, are as follows:]

There are probably few more important questions now occupying the attention of this country—nay, of the world—than that of rendering habitable and populating millions of acres of rich land which has hitherto been labeled "desert" and "arid," owing to the lack of water. This is a magnificent undertaking, worthy of the combined genius of the wise men of the West.

Personally I feel the deepest interest in the question of irrigation. Having spent some twenty years in India, part of the time as a government official responsible for the welfare of the people, and part of the time as a Salvation Army missionary, wearing the Hindoo garb, dwelling in their houses, and intimately acquainted with their daily life, I have had, perhaps, exceptional opportunities for witnessing the immense advantages that lie within the reach of systematic irrigation. In India some of the largest rivers are captured and guided from their mountain sources to their ocean outlet, reclaiming on the way millions of acres that would otherwise be mere desert, and returning in some cases as much as 15 and 20 per cent interest on the capital invested. Not only is the direct flow of the river utilized, but the Hindoo farmer utilizes also to the utmost the underflow by means of wells worked day and night with Persian wheels and bullock power.

One main reason, however, why, in India, irrigation has met with such remarkable success has been the fact that on the border land, and often in the very midst of these irrigated areas, dwells an enormous and purely agricultural community, consisting of small farmers—a

nation of market gardeners one might almost call them—who are ready to occupy every acre of land as fast as it is irrigated, and who have been trained in all the advantages of irrigation from childhood. The movement of population has there been automatic, and has required but little guidance or encouragement on the part of the government.

Our case is somewhat different, and careful consideration must be given to the subject of colonization. Irrigation is dependent for its success upon population. Colonization may be defined as the systematic populating of hitherto unoccupied tracts of land. The lack of systematic colonization has caused the failure of not a few excellent and thoroughly practicable irrigation schemes. The reason for this is not far to seek. A canal which will irrigate, say 100,000 acres of land, is necessarily a costly enterprise. The interest and upkeep can easily be met if the whole tract be quickly occupied. But if a small portion only be settled, either the colonists will be disheartened and driven away by the heavy charges made in the effort to meet expenses and pay dividends, or the investors will become discouraged at the long delay and the nonreceipt of a fair return for their investment. In either case the enterprise will be killed in its initial stage.

Scientific colonization can secure the rapid and immediate enjoyment of the fruits of irrigation. Each is a necessity to the other. An ill-digested and haphazard scheme of irrigation will injure, if not ruin, colonization. Similarly an unsystematic plan of colonization will retard and frequently ruin the best laid and most feasible plans for irrigation.

Scientific colonization will not wait for the farmer with capital any more than manufacture will wait for the laborer with capital. Imagine the captain of industry who would employ no laborer who did not possess a few thousand dollars; and yet this is the course which colonization has generally pursued.

The most that the capitalist, or landowner, or irrigationist has been willing to do in the past has been to bring the water to the land and accept time payments for the latter from the settler. Further than this they have been unwilling to venture; and hence the colonization of irrigated lands has been almost entirely limited to farmers possessing capital.

This has had several serious disadvantages:

1. Irrigation lends itself much more readily to the small intensive farm than to the large ranch from which such settlers usually come.

2. The small farmer who is not too high toned to cultivate the land himself will succeed where the "gentleman" farmer who employs hired labor and sends his children to college will fail, even though the latter may control ten times as much land as the former and possess a few thousand dollars. Give the former a chance, select him with care, and back him with, say, $500 cash for a start, and he will succeed better

than the latter every time. With ample funds for irrigation now in sight, and with some of our brainiest engineers working out extensive plans, what our "arid West" calls for is not the nonresident gentleman farmer, with his staff of cowboys, but the resident, 20-acre, horny-handed son of the soil, who does not consider it beneath his own or his children's dignity to drive the plow, milk the cow, and earn an honest living by his sweat of brain and brow. The old-time governor of Virginia was right when he told King George that he would rather have a dozen such than a shipload of the ladies and gentlemen whom the latter proposed to send out.

But granted that such settlers are desirable, why should there be any difficulty about obtaining them? For the simple reason that they have no capital and can not move themselves.

That there are millions of them prepared to move at a few hours' notice from all quarters of this and other lands in just such numbers as may be desired can not be denied. The talk about their unwillingness to leave the cities or go back to the land is pure, undiluted moonshine. The simple fact is that those who wish to be moved, and who would make the most desirable colonists, are blessed with large families, are often hampered with debt, and are unable to move without financial assistance. Who will pay their traveling expenses, buy their live stock, fix up their cottages, and trust them with land? The farmer, like the manufacturer, wants the single man or woman, and they must be young at that.

Into my office there walked a few days ago a New York banker escorting a splendid specimen of physical manhood—a typical farmer, hale, hearty, manly. His wife was with him—every inch a farmer's wife. He held excellent certificates from an ex-governor of New York and other prominent gentlemen on whose estates he had worked. But with a family of eight children he had been unable to more than barely pay his way. He had no wish for the city; but his large family made him undesirable to the farmer, and he had no capital with which to give himself a start. We made immediate arrangements to settle him on one of our colonies. This is but a typical case among tens of thousands. Not only do our cities abound with them, but multitudes of them may be found in our best agricultural districts. Why do our land agents pass over these laborers in favor of the farmer who has to sell his land in order to move? Not because the latter would make a better colonist, but because he has cash, while the former would require assistance.

Now scientific colonization uses the worthy family that has no cash. It says in brief, "Place this waste labor upon the waste land by means of waste capital, and thereby convert this trinity of waste into a unity of production." It has been argued, on the other hand that, first, they would not go; second, they would not stay; third, they would not work;

and last, they would not pay. We set to work some six years ago to put our theories into practice, and are now able to say positively, after more than five years' experience, that they have gone and stayed, have worked and paid. Even the comparatively few failures we have encountered have been a valuable education to us, and we are now in a position to handle the largest schemes, with self-sacrificing and expert managers to direct them, and with a practical code of regulations to guard us from the rocks on which so many similiar enterprises have been wrecked.

Our three colonies are located in Colorado, California, and Ohio, and comprise nearly 3,000 acres of land, on which about 400 men, women, and children have been settled. On the first two colonies every family is entirely self-supporting, and the repayments have amounted to considerably more than $20,000.

On the California colony last year the settlers averaged a cash income of $850 per family, each 20-acre farm being worth, with its improvements, about $3,000. The Colorado farms are worth from $2,000 to $5,000, according to their location and improvements. On the town site have been established some twenty country stores, most of which are operated by colonists. A commercial club has been formed for the development of the business interests of the settlement. Their turnover last year amounted to about $200,000, while the railroad received some $50,000 for freight from our little country depot.

On the California colony a 30-acre tract has recently been sold for $4,650, including orchard, farm house, and other improvements, being at the rate of $155 an acre for land which cost us some five years previously $50 an acre. I mention these facts to prove that we were not oversanguine when we argued that land thus thickly settled would by its own rapid increase in value amply protect the investor against loss. Thus even supposing that the colonist himself could not or would not pay, the populating of the land would so add to its value that in the course of a few years it could be sold for a sufficient sum to cover the colonist's entire indebtedness and leave him a handsome margin with which to make a new start.

The further extension of colonization will depend, not on land being available, nor on the ability to secure colonists, but on the supply of capital. That this can be safely invested we think we have sufficiently demonstrated.

That there is land in abundance, admirably suited for colonization, no one will deny. The new irrigation projects now on foot will make available immense regions with fertile soil and salubrious climate, suited in every sense to be converted into a veritable poor man's paradise.

What a magnificent outlet is here afforded for the lamentable congestion of our great cities. How unspeakably sad, how incredible must it appear, that in this great, rich, prosperous country of ours

no less than 3,000,000 of our fellow-citizens—men, women, and children—at least 50 per cent of them agriculturists—should be "cabined, cribbed, confined" in the unhealthy tenements of our cities, while there exist at the very gateways of our civilization these boundless prairies with their invigorating breezes.

President Roosevelt spoke to the heart of the nation when he pointed out the dangers of race extinction and the importance of the family—the large family—to the well-being of the nation. The pivot of true social reform appears to me to turn upon the preservation and, if necessary, the restoration of the family unit. Destroy the home and you destroy the nation. We must show the poor man how he can afford to get married and can bring up his family in decency and comfort, and become a home owner. If this can not be in the city, let us throw open to him our irrigated lands and provide him with the means for making a start. There is no need to treat him as a pauper. We can charge him with every dollar that we expend upon him, and make him pay a reasonable interest into the bargain.

How much better would it be if the fifty to one hundred million dollars now expended annually in the relief of poverty in our great cities could be devoted to the placing of worthy families on these rich irrigated lands. Tax consumers would become tax producers. Pauper dependents on charity would be converted into affluent home owners. The $50,000,000 annually expended for charitable objects would be turned into a rich investment which would be amply secured and ultimately repaid out of the profits from these irrigated lands. In ten years the money thus invested would amount to $500,000,000, which would suffice to locate 5,000,000 souls upon, say, 20,000,000 acres of irrigated land, which would produce to its owners an annual income of, say, $400,000,000, while the value of the land at $50 per acre—a moderate estimate for land so closely settled—would be no less than a billion dollars.

What our American poor ask for is not charity but opportunity. Let this congress then throw wide open before our working classes that door of opportunity. Let it place within the reach of the landless man our manless lands. Let it speak with a voice which can not be misunderstood, and let it thunder at the doors of our national Capitol and of our State legislatures till colonization is made as much an imperative and practical question of the hour as by its brilliant and persistent efforts irrigation has already become.

The association of these two powerful factors, irrigation and colonization, in the welfare of our nation will be well-nigh irresistible, and with these mighty levers this congress will lift from the nation's pathway the dead weight of poverty and congestion which has obstructed our national progress, created internecine struggles between capital and labor, and threatened to shipwreck our future prosperity.

WORK OF THE RECLAMATION SERVICE.

By Hon. FRANK W. MONDELL.

I very much appreciate this opportunity of meeting the engineers of the reclamation service and of hearing your discussions of the surveys and examinations of the various projects contemplated under the national irrigation law. I wish to congratulate Mr. Newell on having surrounded himself with so able a corps of engineers and lieutenants, and I wish to congratulate all of you on your employment in a service so important and beneficial and which promises so much for development by irrigation in western America.

I think the idea of a gathering of the engineers of the reclamation service at this time for the purpose of consultation and discussion is a very happy one. The mutual exchange of views and opinions will undoubtedly result in much good, and by comparison of the respective advantages and difficulties of the various projects under consideration each of you will have a better idea of the relative merits and feasibility of the particular project on which he is engaged.

It would be clearly superfluous for me to enter into a discussion of the national irrigation act with gentlemen who are so thoroughly informed in regard to it as are you who are charged with the practical work of carrying the provisions of the law into effect. I venture to say, however, that in your operations under the law so far you have been struck with its broad and comprehensive character. It was not intended, as we all understand, that the act or the operations under it should interfere with, supplant, or to any considerable extent curtail the legitimate field of private enterprise in irrigation development. My observation is that the effect of the law so far has been to encourage and stimulate private enterprise all over the arid region. And the territory to be covered is so vast, the possibilities so great, and the conditions so diversified, that we must needs have the active aid of every possible agency, private and public, to accomplish the complete reclamation of the arid regions of our country.

The field of operations of the reclamation service is vast, and in my opinion will be a constantly growing one; that is to say, projects which at this time seem out of the question by reason of their great cost will, in time to come, as the demand for irrigated land increases, be undertaken, and this will undoubtedly include a wider variety of projects than are in contemplation at this time. Primarily the national irrigation act contemplates the irrigation of land now in public ownership and upon which new homes can be established, but its provisions are clearly applicable to the irrigation of lands in private ownership, with or without a partial water supply, and in some cases the best results will undoubtedly be obtained by works which will supply water for lands so situated.

I am one of those who believe that in carrying on the work of the reclamation service you will encounter much less difficulty than has been generally supposed in adjusting the matter of water rights on interstate streams. The supreme court of my State, Wyoming, has recently handed down a decision, which, in my opinion, will come to be recognized as the universal law of the arid region, to the effect that priority of use gives priority of right to the use of water on an interstate stream, regardless of State lines. With the improvement of water laws in the various States this principle can be easily carried into effect.

The crowning virtue of the national irrigation act lies in the fact that it compels the division of the country irrigated into small farms, and therefore insures that thorough and intense cultivation which is essential to the success of any irrigated region.

INVESTIGATIONS IN ARIZONA.

By Arthur P. Davis, Supervising Engineer.

The irrigation project contemplated for first construction in Arizona is a storage reservoir on Salt River at the mouth of Tonto Creek. It will directly irrigate about 150,000 acres of land near Phoenix, the capital of the Territory, and will furnish power for the irrigation of possibly 50,000 acres more by pumping from wells. The proposed reservoir site is very nearly inaccessible, being remote from railroad connections and surrounded by rough mountains and profound canyons.

Borings were made on this site in 1901 at the expense of a water-storage commission that had been organized under the laws of the Territory of Arizona and had been provided with funds raised by taxation in the county of Maricopa, which includes all the irrigable lands. The borings proved the foundation and all conditions to be favorable. On the dam site finally adopted the deepest hole to bed rock is 28 feet, and the rock is of the most excellent character. Samples of the rock were crushed in the crushing machine of the navy-yard at Washington and showed higher tests than average granite used for building purposes. In fact, the machine used for the testing had never been put under such a strain before.

From the direction of Phoenix there is no road except one that is very roundabout and very long. There is a very rough trail through the country, which can be followed with difficulty. The easiest method of gaining access to the dam site is by leaving the main line of the Southern Pacific at Bowie and there taking a branch road, about 125 miles long. This road has no competitors and its rates are exceedingly high. At the former freight rates and prevailing prices the cheapest first-class cement that could be laid down at the end of the

branch would cost $7.89 a barrel, and the additional wagon haul would bring the cost still higher.

For a structure requiring over 200,000 barrels, the cost of cement alone is immense. To avoid bringing in the great quantity required, search was made for cement-making material near the dam site, and it was found that materials suitable for the manufacture of Portland cement can be obtained in the vicinity. These materials were first tested in 1901 by the chemist of the cement works at Colton, Cal., who, after a thorough trial, reported that they were suitable for making a true Portland cement of a superior quality. To check this determination further tests of the material were made by a firm in Philadelphia, and their analysis also shows that it may be used for manufacturing a first-class article. It has been found that the building and machinery for a cement plant there will cost about $120,000. The actual cost of manufacturing the cement, after constructing the mill and providing power, will be about $2 a barrel. By adding this to the cost of the plant it will be seen that the cement will cost $2.50 a barrel.

We have also been handicapped in the matter of power. There is very little fuel in the valley, just a little scrubby mesquite and sparsely scattered cottonwood. We would be seriously handicapped in getting fuel into the country because of the poor transportation facilities. To overcome this difficulty it is proposed to develop water power in the river. The water can be diverted through a canal about 18 miles long, and ample power can thus be developed for all purposes for which power can be used in the construction of a large work of this kind, and afterwards used for pumping.

A unique feature of this project is the proposal to utilize the power generated by drawing water from the reservoir under full head through water wheels and transmitting it a distance of 60 miles to the valley below for power purposes.

Fifteen years ago the highest dam in the world was the Furens dam, in France. Its total height was about 170 feet. Since that time three structures in the United States of very much larger dimensions have been commenced and are nearing completion. One is for the city waterworks of New York, on the Croton River; another is for the waterworks of Boston, at Clinton; the third is for the waterworks of Denver, on the South Fork of the South Platte River. These dams are all much higher than the Furens dam in France. Each one will in some respect hold the record for high dams; the Croton dam will be the largest mass of masonry; the dam at Clinton will impound the largest quantity of water, and the dam on the South Platte will be the highest. When completed, as proposed, the dam on Salt River will be the highest dam in the world, and will impound twice as much water as all

these three reservoirs put together. It will be about 270 feet in total height and will impound 1,200,000 acre-feet of water, forming the largest artificial storage reservoir yet constructed.

SALT RIVER VALLEY WATER USERS' ASSOCIATION.

By B. A. Fowler.

[Mr. B. A. Fowler, president of the Salt River Valley Water Users' Association, described the steps leading up to the organization of this association and the difficulties encountered.

The Salt River Valley Water Users' Association is the first organization created to operate under the terms of the reclamation law. Section 6 of that law states that "The management and operation of the irrigation works shall pass to the owners of the land irrigated thereby, to be maintained at their expense, under such form of organization and under such rules and regulations as may be acceptable to the Secretary of the Interior."

Where there are considerable bodies of land in private ownership, and where a large number of persons must be dealt with, it is evident that the earlier organizations like those contemplated by the act are created, the more definitely will the work proceed. It is necessary, for promptness of action with these persons, to have a common agent or representative to transact business. In order, therefore, to bring about the construction of the Salt River dam in Arizona the above-named organization has been formed. Being the first of its kind, it was necessary to consider many matters which subsequent organizations will probably adopt without further thought.

The articles of incorporation, resulting from much deliberation and correspondence, are presented in the following pages. It is thought that these cover most of the requirements of law and good administration.]

CONDITIONS IN SALT RIVER VALLEY.

The first irrigation ditch in Salt River Valley, Arizona, was dug in 1868. As the population increased other ditches and canals followed, until there are now about 14.

The early settlers had water in abundance, and no one then anticipated a scarcity; but as from time to time additional land was put under cultivation it became evident that the use of more economical methods, the better conservation of the normal supply, and the impounding of winter floods were of vital importance. Several unorganized and short-lived movements to procure a reservoir or some system of water storage for the valley were made prior to 1900, but these accomplished nothing except to arouse a temporary interest in the subject.

In April, 1900, the valley was beginning to feel the effects of a drouth which had continued for nearly two years with increasing severity. At that time the Phoenix and Maricopa County Board of Trade appointed a committee of five members to investigate and report upon conditions and needs and to devise ways and means for providing a water supply sufficient for the purposes of irrigation. This committee published a report the following month, which was widely circulated throughout the valley, and led to the calling of a mass meeting of the landowners on Friday, August 31, 1900, in the Grand Opera House at Phoenix. The auditorium was filled with an eager and interested assemblage of landowners from all parts of the valley, who continued in session all day.

As a result of this meeting a water-storage committee of 25 members was selected, each canal being represented, to which was added the five members of the board of trade committee. Immediately upon the adjournment of the meeting this committee organized, and on the following Monday held a meeting, at which a committee of five was appointed by the chair to nominate subcommittees to investigate and report upon various subjects connected with irrigation. Frequent meetings were held by the executive committee, and diligent investigation and correspondence were begun wherever the different committees thought to gain reliable information which might be utilized.

The interest of the entire valley had by this time become aroused to the paramount importance of the water question, and it was felt that far-reaching legislation on this subject would in all probability be enacted by the next territorial legislature. Accordingly, the friends of water storage, in November, 1900, elected representatives to that body in sympathy with the movement, and also sent a delegate to the National Irrigation Congress, held at Chicago, Ill., the same month.

SURVEYS IN THE VALLEY.

About this time a letter was sent to the Secretary of the Interior by Mr. B. A. Fowler, as president of the Salt River Valley water-storage committee, setting forth the conditions existing in the valley and the action taken by the residents, and offering on their behalf to contribute the sum of $1,500, to be expended for investigations if an equal amount should be allotted from the funds of the United States Geological Survey.[a]

This letter was referred to the Director of the Geological Survey, who replied to the Secretary of the Interior on January 14, 1901, recommending the expenditure.

Hydrographic work was pushed on Verde River through the winter and later in the Tonto Basin on Salt River. In the meantime an

[a] See Water-Sup. and Irr. Paper No. 73, U. S. Geol. Survey, 1903, p. 10.

act was passed by the Arizona legislature, then in session, creating a "water-storage commission," providing for a tax levy of about $14,000 a year for two years for purposes specified, and authorizing the commission "to cooperate with or contribute toward the expenses of any investigations now being made or hereafter to be made by the United States Geological Survey."

The hydrographic and preliminary work continued throughout 1901 and into 1902. Up to this time efforts had been concentrated on procuring all reliable data obtainable before deciding how best to secure construction. There were many plans and schemes, but only two met with much favor. One was to make it an object for private capital to construct, the other was that the county should build and issue bonds in payment for the work. Both ways were beset with difficulties and risks, and each met with decided opposition. There were a few who dared hope for Government aid, and firmly believed that national irrigation was near at hand. But by the great majority these were called "rainbow chasers," "dreamers from dreamland," and considered a little "off."

PASSAGE OF THE RECLAMATION ACT.

Meanwhile the national irrigation movement was steadily growing, the country over, and finally crystallized into the reclamation act of June 17, 1902. Under it the people of Salt River Valley, or many of them at least, realized that they might become its beneficiaries, and without delay began to put themselves in harmony with its requirements, as they interpreted them. There was no official interpretation, no court construction, no authorities to be consulted. The only guide was the law itself—new, untried; a magnificent experiment.

MEETING AT PHOENIX.

A mass meeting was called in the court-house at Phoenix, August 2, 1902, at which was chosen a committee of 25 citizens, representatives of the various canals, to be known as "The Water Storage Conference Committee," which was organized one week later. At this meeting an invitation was extended to Mr. George H. Maxwell, executive chairman of the National Irrigation Association and the active exponent of its work, to come to the valley and assist in formulating a plan in accord with the national irrigation act. Responding to the invitation, Mr. Maxwell came and addressed a large meeting of farmers and landowners August 9, 1902, and outlined a plan of action, which met with general approval.

JUDGE KIBBEY'S ADDRESS.

Early in September, 1902, Judge Joseph H. Kibbey, of Phoenix, author of the well-known "Kibbey decision," presented to the com-

mittee for consideration a plan, comprehensive in scope and rich in practical suggestions, drawn from a wide and varied experience of many years in dealing with such matters. In the statement of his plan Judge Kibbey said:

The earliest legislation in this Territory recognized the right of appropriation of water from its streams, lakes, etc., for, among other things, the cultivation of the soil, and recognized that appropriators are entitled to priority of use of the water in the order of the dates of appropriation, and, I think, by necessary inference, the right is appurtenant to the land for which it was appropriated, as the right was limited expressly to the owners or occupants of the lands.

In 1877 Congress, pursuing the long-settled policy of having actual tillers of the soil as the owners of the national domain, provided by the enactment of the "desert-land law" that the entryman could acquire title by paying a nominal sum per acre for the naturally desert land, and authorized him to appropriate water from the public sources of supply for its reclamation and cultivation. This law gave priority of right to such appropriators in the order of the times of appropriation, and upon proof of such reclamation and the permanent right to the use of water the Goverment transmitted to the entryman, by the same deed, a title to both land and water.

The latest act of Congress, the Newlands Act, contemplates that stored or other water impounded or developed under its provisions shall be used only by those acquiring the right to use it under the act, and that the right shall be inseparable from the land. Thus we have defined the status of the landowner who has a right to the use of water.

The acts constituting an appropriation of water are held to be—

(1) The diversion of the water from the source of supply.

(2) The conveyance of the water to the place of use.

(3) The actual use

These different acts may be, and in practice in this valley are most frequently, performed by two different persons; that is, the diversion and carrying or conveyance is by one (an association or a corporation usually), and the actual use by another (the landowner or occupant). The landowner or occupant is the appropriator, for by the law he is the only one who can appropriate water, and the carrier is merely an agency adopted by the appropriator to effect the appropriation. The relation of the one, the appropriator, to the other, the carrier, must arise by contract, express or implied, or, if the term may be used to promote brevity of expression, by estoppel.

Now it is probable that of these propositions none will remain undisputed by some of those whose interests are, or seem to be, variant. The first, or, in any event, the most essential thing to be done, is to settle these disputed questions, and if success is to be attained it is submitted they must be settled as here stated. To settle them by simple agreement is impracticable, for that involves unanimity of all the disputants, which is hopeless; and it would be difficult to frame an agreement so definitive of the rights of all the parties that it would be satisfactory to either the Government or to the parties themselves. Nothing short of a decree of a court of competent jurisdiction will suffice, for that alone can be either definitive or final.

A very casual consideration of the Hansbrough-Newlands Act will make apparent the absolute necessity for this settlement. It may be assumed that, from motives of self-interest—which in any case are always the safest to which to appeal—if from no others, the appropriators and the carriers of water in this valley urgently desire a prompt, definite, and final adjustment of these questions.

The variant interests may, in a broad way, be divided into two classes; that is, the water users and the carriers, where the user is not also the carrier. There will have to be settled among the one class, the users, the question of time and extent of

appropriation of water, and as to them, the fact that the right to use water is appurtenant to the land for which it was appropriated. Among those of the other class, the carriers, assuming that they may preserve their autonomy, there will arise questions of their relative rights to divert and carry water. All these questions are preliminary to and independent of any consideration of the administration of stored water, and are questions in the determination of which the Government probably can not and certainly will not engage. They must be settled before we can go to the Government at all. It may be taken for granted that the Hansbrough-Newlands Act will be construed to authorize the construction of reservoirs for the storage of water for the irrigation of lands now in private ownership for which there is only a partial and hence an inadequate supply. This construction seems easily warranted, and is in entire consonance with the purposes of the act.

The act also contemplates the organization of the owners of the lands irrigated by the reservoir or other works into an association for the management and operation of the works; but this management and operation will not, under the terms of the act, vest in such association until the major part of the cost of the works shall have been repaid to the Government.

The cost of the reservoir at Tonto (or practically all of it) will have to be assessed against the land that is held in private ownership, at a rate per acre to be fixed by the Government. This assessment will be divided into annual payments extending over a period of not more than ten years, and beginning at a time to be fixed by the Secretary of the Interior, probably when the reservoir is ready for service.

The act suggests that the Government will provide its own means for enforcing payment of these assessments, except possibly, and perhaps preferably, if the plan of association hereafter proposed shall be approved by the Secretary of the Interior and the association selected as the medium of the collection of these payments to the Government.

Now, if a plan can be devised whereby, first, all disputes among the variant interests concerned in the storage project may be settled so finally and definitely that the Government may have no concern in that respect; and, second, the owners of land now in private ownership and homesteaders under the act shall initiate rights to the use of the stored water in the manner outlined in the act in such numbers as to warrant the Government in the belief that sufficient money will ultimately be repaid to reimburse it for its expenditure in the construction of the reservoir, then we have brought ourselves within the purpose and scope of the act and subjected ourselves to its conditions and have reason to believe that our application will be favorably entertained by the Secretary of the Interior.

To accomplish these things it is proposed to at once organize the association of water users contemplated by the act itself. They should incorporate under the laws of the Territory of Arizona. Those, and those only, who have initiated rights, or may initiate rights to the use of water from the reservoir, should become constituent members. The act defines those to be homesteaders or owners of lands now in private ownership, who initiate rights to stored water under the provisions of that act.

The capital stock should be a sum equal to the estimated cost of the reservoir and other works undertaken by the Government and should be divided into a number of shares equal, at a conservative estimate, to the number of acres that may be so supplied with stored or other water. The division of the entire fixed capital by the number of shares so ascertained will of course fix the par value per share of the stock. As the proposed stockholders can not now initiate their water rights under the reservoir, and can not do so until the Government shall offer them for acquisition, the subscription for the stock should be for a cash payment for it at par, with a provision in the articles of incorporation, made irrevocable except upon proper consent of, say, the holders of three-fourths of the stock; that the stock of any sub-

scriber shall be deemed fully paid up upon the production by him of proper evidence that he has initiated a right under the reservoir, conditioned for forfeiture for his failure to comply with his engagement with the Government, or with the association as the agent of the Government for that purpose, in like manner and under the same circumstances as imposed by the Government, and that until that time the subscribers of the capital stock subscribed for shall not be subject to be called upon for payment of their subscriptions.

In no event does the proposed corporation deal with the Government in its corporate capacity in initiating or perfecting any individual right; all those negotiations are by the individual with the Secretary of the Interior, precisely as though there were no such corporation, or as though, if there were one, the individual was not a member of it.[a]

The initiation of the acquisition of the right and its final perfection are mere qualifications to membership of the proposed corporation, and it is the principal qualification required by section 6 of the Hansbrough-Newlands Act for membership in such an organization.

In making estimates the greatest care and the most conservative course should be taken. If error is made in fixing the estimate of cost too high and the extent of acreage of land too low they can be corrected to meet the event with advantage to all; while if the estimate of cost is made too low and estimate of irrigable acreage too high the mistake can not be remedied. To exceed the limits ultimately and irrevocably fixed by conditions in the estimate of lands capable of irrigation by the reservoir is disastrous. We can expand to meet conditions; contraction is impracticable, if not impossible.

Until the management and operation of the works shall be turned over to the associated land owners having rights under the reservoir, the Government will of course have the administration of the stored water; and that, it may be assumed, will be in accordance with expressed wishes of the right owners whether expressed by them jointly through their association or by them individually. A partnership of Government and association is not proposed, nor can there be interference by the association, either with the purpose of the Government or its methods. Both are independent, seeking to accomplish a common purpose, with the association always subordinate to the Government, until by lapse of time and in the course of events, the management and operation of the works, by the terms of the law, vests in the association.

Now, it may happen that some may subscribe to these articles of incorporation who may never qualify by becoming the initiators of rights under the reservoir, either because they do not apply to become so or because there may not be enough rights to be had. To meet this condition a provision should be made that applications can be made for such rights within a reasonable time, failing which the subscribers shall become disqualified, shall be released from his cash subscription, and shall lose his rights as a stockholder.

Incident to the power to manage and operate the work constructed by the Government as provided in section 6 of the act, are naturally the power to acquire the means of distribution of the water stored and developed—as waterways, pipe lines, flumes, gates, dams, etc.—and to acquire or use agencies for that purpose, and to manage and control any of these, according to the rules prescribed by the Government as to the stored water, pending the vesting of that management and operation in the association; and independent of that, but of course never inconsistently, to distribute other than stored water, as long and in so far as a distinction can be and

[a] Since writing this Mr. George H. Maxwell, in his address to the people here on August 9, 1902, suggested that the Government would probably prefer to deal with the association rather than with individuals who seek to acquire rights under the reservoir. If so, that would simplify matters, and the association could become practically the agency of the Government in that respect.

is desired to be preserved between stored water and that normally flowing in the rivers. Those powers should be carefully defined and stated in the articles of incorporation and yet made comprehensive enough to meet any conditions existent or that can be anticipated.

There are now waterways and other means of distribution in the valley more or less adapted to the needs of distribution of stored as well as other water. Provision should be made for the use of these means, or such of them as may be expedient, and also, if after careful consideration it be deemed best, for the acquisition of new and other means. It seems almost too flat a truism to say that the Government will not undertake to store water if there are no means provided for its distribution to the users—and those means must be adapted to that purpose. It is true that the act may be construed to authorize the Secretary of the Interior to construct or otherwise acquire means of distribution of the stored water, but it seems unreasonable to expect it, under the conditions prevalent in this valley.

As the means for the distribution of stored water is so necessary an adjunct to the storage works, the Government will naturally and logically require that the means of distribution shall be coextensive with the storage works themselves, entirely adequate to the service, and subject irrevocably to all the rules of practical use necessary to accomplish the purpose of the construction of the storage works—i. e., the use of the water so stored.

The owners of the present canals have certain rights—property rights—of which they can not be divested without their consent except by the exercise of the power of eminent domain. Yet it is extremely desirable that the means of distribution of all water, stored and other, shall be under the immediate and exclusive management and control of the water users. Several plans suggest themselves for the accomplishment of this desideratum.

If the canal companies, the owners of most of the canals, should preserve their autonomy and the ultimate title to their property, then a perpetual contract could be entered into between them individually with the association of water users, whereby:

First. The property shall be perpetually leased to the association for an agreed reserved rental, payable annually or otherwise, with a right to the association to purchase; or

Second. The canal company shall undertake to carry and distribute stored and other water to its customers at a reasonable cost, to be agreed upon between the company and its customers, subject, however, to the rules of the Government as prescribed by it for the distribution of stored water, and to the rules prescribed by the association for the distribution of both stored and other water, not inconsistent with those prescribed by the Government, with an agreement to sell to the association.

Either of these plans fall short of the desideratum, and involve many difficulties. Still they are both practicable. Each of the above propositions, it will be noted, reserves to the association the privilege of purchase. Many contingencies, too numerous to mention here, but necessary to be considered, are to be met, as expiration of corporate franchises, existing contracts, mortgages, etc.; difficulties not insuperable, but tedious and complex in the working out.

A more desirable plan involves the purchase outright by the association of such of the canals and other works as may be found necessary or desirable for the purposes of the association. A rather anomalous situation would arise between the proposed association and the canal companies considered as possible purchaser and seller. There would be but one possible purchaser and that purchaser would be desirous to buy, and there would be but one seller and he would be desirous to sell; it would seem, therefore, that they should easily agree upon terms.

The maximum value of the property to be sold is simply the amount of money necessary to construct a new system of canals, etc., adequate to the carriage and distribution of both stored and other water.

If this plan of purchase were adopted it would have to be after it was found to be desirable—after negotiations had progressed so far as to determine the price and the terms of payment. The price should be satisfactory and the terms of payment should be so adjusted as to be conveniently met by the purchaser. In this plan, as in the others suggested, there will be difficulty in adjusting the rights of lienors and others who will have to be dealt with before final consummation. The rights of water-right owners offer no obstacle, for if the association is formed as suggested, and a purchase of any or all of the canals are effected by it, the water-right owner's right is replaced by a better one in the association and merged in it, as will be hereafter explained.

Another plan, and the simplest, is to acquire the canals by the exercise of the power of eminent domain—by condemnation proceedings. This plan may be adopted to acquire the title to one or more of the canals, or of all of them seriatim, as the association may elect, the procedure being entirely adequate to those ends. This plan eliminates in a great measure a consideration of the right of lienors, etc., as they are transferred to the fund created by the payment of the assessed damages.

Finally, the association may construct its own system of canals and other distributing works.

The cost of the several plans that may be considered might and should have considerable influence in the determination of the selection of a plan.

Now, as the raising of funds for meeting the cost under any plan: The articles of incorporation should provide that all the ordinary expenses of maintenance and operation should annually or at other stated intervals be estimated by the directors and assessed pro rata by the number of acres, against the stockholders, and that when an extraordinary expenditure is contemplated, as for the purchase, construction, or other method of acquisition of a canal or other works, exceeding in cost, say $5,000, the board of directors shall formulate a proposition for the same to be submitted to the stockholders. The proposition should contain a statement of the purpose of the contemplated expenditure and the estimated amount of it, full enough to acquaint the stockholders in a general way of the purpose of the expenditure. Upon a resolution of two-thirds of the directors a special meeting of the stockholders should be called to consider the proposition submitted, and if holders of two-thirds of the stock should vote in favor of the proposition the directors should proceed to carry on the work proposed, and should assess the estimated cost thereof against the stockholders, as hereinafter proposed.

For the purpose of these extraordinary expenses the country embraced in the lands under cultivation by stockholders of the association should be divided into districts. These districts should be made severally to consist as nearly as may be practicable of lands immediately supplied by the canal system; as, for example, the lands under the Salt River Valley canal should constitute one district, those under the Maricopa another, those under the Utah another, and so on. Then when it is proposed by the association to expend money for an extraordinary purpose, as for the acquisition of a canal by purchase, condemnation, or construction, the cost should be assessed against the stockholders of the district particularly benfited, and among those in that district as they are individually benefited.

It should be provided that if in particular cases it should seem inequitable to assess the whole cost against a particular district, then the council should have power to direct what portion should be so assessed, and what portion should be borne by the association at large.

If the proposed expenditure is not all required to be made in one year the assess-

ments may be made payable in annual or other periodical installments, adjusted so as to meet the expenditure as they may be required. The manner of enforcing the payments of such assessments is set forth in another place.

The articles should provide that the shares of stockholders should be complete evidence of the right of the holder to the use of the water for the irrigation of designated land at the rate of 1 acre for each share of stock, subject to the rules and regulations prescribed by the association therefor; that the ownership of stock should be evidenced by a certificate thereof, duly attested by the proper officers of the association, wherein the lands to which the water right is appurtenant are described with certainty sufficient to identify them; that the stock should be transferable on the books of the association, but as appurtenant to the lands described in the certificate, and the same should be deemed to pass, whether expressly conveyed or transferred by any conveyance of the land, or pro tanto any part of it, to the successor in title, whether such transmission of title is by grant or by operation of law. It should be further provided that any stockholder by the act of subscription shall pledge the land and appurtenances for which water rights are acquired and evidenced by such certificate of stock for the prompt payment of all assessments made in the manner prescribed by the articles, and that the lien for the payment thereof may be enforced by a sale of the land or so much thereof as may be necessary to discharge the same from time to time, and as often as such assessments are in default, such sale to be by the treasurer of the association, upon due notice, and at a place designated, with a reasonable time of redemption reserved. The particular provisions concerning the enforcement of the collections of the assessments should be very definitely stated.

The creation of this lien will probably require the assent of the wife where the land is community property. It must be remembered that the whole matter, whatever plan be adopted, will, particularly at first, involve long and tedious labor. But there is no evading it, and it ceases to be formidable when it is considered that the work is no more difficult than the assessment of county taxes.

Each stockholder of at least one share should be allowed one vote at all corporate elections, and only one, whatever the number of shares he or she may own, and his or her qualifications as an elector should be determined by reference to the association's books as they appear, say, twenty days before an election. In no event should voting be permitted by proxy, but should be done at the designated polling place or places, by the stockholder in person.

Regular elections should be provided for, to be held annually for the election of directors and for the transaction of such other business as may be properly referable to the stockholders. These regular elections should be held in the spring, so as to be free from the influences of and the confusion incident to county and territorial elections. Specific rules governing elections should be provided, as for polling places, election boards, voting, canvass of votes and returns, etc. Provision should be made also for the calling and holding of special elections. There should be provided a council, in whom should be lodged the general legislative powers of the corporation as to making, modifying, or repealing by-laws for the government of the members and the directors. The council should consist of, say, 40 members, who should be stockholders, one-half elected every year, and holding two years. This council should meet in regular session at designated times and places, and specially when called by the directors, or by a designated number of the stockholders. To this council should be left the general matters of the by-laws, the fixing of salaries, and compensation of officers, etc. They should serve without compensation and be governed in the transaction of their business by rules usually governing legislative bodies.

The immediate management of the affairs of the association should be vested in a board of, say, nine directors, and in such officers as the articles of the association or

the council may prescribe. The board should hold regularly designated meetings and special ones in a manner to be prescribed by the by-laws. The directors should be allowed a proper compensation, to be fixed by the council. There should be a president, with the usual executive powers of that office, who should also be the presiding officer of the council. There should also be a secretary and treasurer exercising, respectively, the usual functions pertaining to those offices, who should be required to execute a proper bond in amount and with sureties prescribed by the council. Any proposition emanating from the directors that may involve an expenditure of more than $10,000 should be first approved by the council before it is submitted to the stockholders. The council may prescribe penalties for the infraction of by-laws and provide that those imposed, if not otherwise discharged, shall be added to the assessments against defaulting stockholders and collected as assessments are.

A discussion of the rights of water users here seems unnecessary. That there is a priority of right to use water must be conceded. Priority of appropriation is the fundamental rule of all property; the first right to it was its appropriation, and every right is traceable to it. As with any other species of property, so it is with the property in the right to use water. If there is enough for all to take indefinitely, and as it is wanted, the exercise of the right by one person conflicts with that of no other, and priority becomes a distinction of no practical importance.

If the reservoir is built and fulfills our expectations there will be no distinction in practice in the distribution of stored water and other water, whatever its source, by the general reservoir and canal system. If it can be obtained, there will be applications for water for the 1,000,000 or more acres susceptible of service. But there will not be enough, there can never be enough, to meet the demand.

Then how shall applicants be served? It is the most logical, the most natural, the most just, and the most practicable plan to meet the demands in the order of the existing appropriations until they are supplied, and if there be a surplus then it may be supplied to new lands acquired by homesteading under the provisions of the New-lands law.

Enough has been said to demonstrate the necessity of speedily and finally settling the question of priority of rights of land owners within the territory proposed to be supplied by the Tonto reservoir. Two plans suggest themselves, the selection of one or the other much depending upon the readiness with which the proposed beneficiaries approve of either.

One plan requires the agreement of all the water users to submit the question of the date of appropriation, the extent of the appropriation, and manner of use to arbitrators selected under the statute for award with a provision that the award shall be final, without appeal, and that the award be made a decree of court.

The other plan is to institute a suit in the district court, having all the waters users parties plaintiff or defendant, with the incidental right of omitted parties to make themselves parties; then to have the case submitted to a referee to report to the court his conclusions as to time and extent of the appropriation of each party, and in due course of time to obtain a decree of the court thereon.

It should be made a condition precedent to the acceptance of a subscription to the capital stock of the proposed association heretofore outlined, that the subscriber shall have agreed to submit these questions for settlement as proposed, or that the extent and time of his appropriation shall have been determined by one or the other of these proceedings. After the determination of these rights the owners of them should be classified, each by the year of his appropriation.

When the Government is ready to issue rights under the reservoir, preference should be given to applicants therefor according to the class in which their appropriations are found, preference being given to the oldest, and thence in order of time to the newest.

Reasonable limitations of time should be fixed within which this preference may be exercised, imposing upon a delinquent the penalty of postponement to await his turn at the last.

It is of course contemplated that after the plan here proposed shall be in operation there will be no pecuniary benefits accruing to the stockholders by way of profit of operation, and that when the cost of maintenance and operation has been provided for there can and will be no further charge to the water user.

The association will have no interest-bearing debt to be charged up against the water users. The cost of management and administration can be reduced to the lowest possible limit by concentration and systemization. Taking these things into consideration it is conservative to say that the cost of water to the user ought not to exceed 75 cents per acre per year. And this cost can be still further reduced from various sources of income.

The water impounded in the reservoir will afford power at a cost of one-fourth of that now paid for power in this valley, and even then produce a handsome income.

The cities and towns in the valley will need a supply from which an important income can be derived. Power could be transmitted to pump water for the Pima Indians and the requirements for that purpose would hardly make an appreciable diminution of the whole amount developed, and would afford them a greater, more certain, more constant, and a more reliable supply of water than they ever had. The power could be used to raise the water now stored and constantly renewed in the gravelly strata underlying the surface of the valley—an amount of important magnitude.

There are dangers, too, some of them already realized, that we will be better equipped to confront. For more than fifteen years there has been a constant diminution of the water flowing into this valley. Aside from the unusual meterological conditions that have been prevalent for the last three or four years, this has been due to the constantly increasing diversion of water from the tributaries to our supply. And it is going on unchecked. It would be one of the most important functions of the proposed association to check this encroachment. That no attempt has heretofore been made to remedy this sufficiently suggests that while the present conditions of diversity and antagonism of interests continue in this valley it need not be expected. With unity of interest it may be hoped for. We can not console ourselves with the hope that matters with us can not grow worse than they now are. Simple inactivity will make them worse. Whether the Government builds or aids in building the Tonto reservoir or not, to stay our progress to a disastrous end there must be a unification of the interests of the water users of this valley.

The plans and suggestions here submitted are tentative. The attempt has not been made to work out every proposal in detail. The desired result has been anticipated in each instance, the general means to the end has been suggested, and the filling in has been left for later time and for more varied and thorough deliberation.

If the plans and suggestions do not, or some of them do and others do not, meet with approval, yet if they have the effect of pointing out other and better ones, this paper will have justified itself.

WORK OF THE EXECUTIVE COMMITTEE.

An executive committee of eleven having been appointed, of which B. A. Fowler, chairman of the Water Storage Conference Committee, was made chairman, a general plan for the organization of the landowners and irrigators of the valley into a cooperative water users' association was unanimously agreed upon and reported to a mass meeting held October 4, 1902, where subscription books were opened for

what was to be known as a "preliminary" subscription approval of the plan. This carried with it no financial obligation or legal responsibility whatever. Its purpose was solely to procure from landowners an expression of willingness to "sign up," and by so doing to inaugurate a campaign of education which should reach every portion of the valley and thus bring about a better understanding of the whole proposition by the masses of the people.

The work of procuring "preliminary" subscriptions was pushed systematically and vigorously, until about 150,000 acres had been "signed up." Meanwhile the regular meetings of the executive committee were resumed, and Judge Kibbey was employed as legal adviser of the committee.

To Mr. Maxwell and Judge Kibbey is due much of the credit for the character and quality of the work finally embodied in the report of the executive committee. Each supplemented the other, and, to the finish, they worked in perfect harmony, safely guided by their sound sense, legal training, and wide experience in irrigation law.

Frequent meetings of the executive committee were held during November and December, 1902, and January, 1903. It was a comparatively simple proposition to reach a general outline agreement in October, but quite another to blaze a new trail—to formulate, without precedent or guide, articles of incorporation which should be broad in scope, clear in statement, carefully worded and legally phrased, just to all interests, and adapted to existing conditions. Of course, among eleven men of strong individual characteristics and personalities there were radical differences of opinion, views held tenaciously, much discussion (rather heated at times), and great deliberation. But, to their everlasting credit be it said, there was no wrangling, no ill-tempered outbreaks, no attempt at sharp practice. The minority was of tough fiber, active, alert, everlastingly persistent and aggressive, but in the main fair, open to conviction, and amenable to just rulings, and at the last meeting of the committee joined in giving the chairman a unanimous vote of thanks for his fairness and unvarying courtesy, at times under very trying circumstances. The entire committee realized that only by "infinite patience and justice to all," as Mr. Maxwell put it at the first meeting, could success ever be attained and antagonistic interests be harmonized and unified—an achievement declared by many in the earlier days of the movement to be utterly impossible.

The opposition of the minority was due, first, to a reluctance to abandon old and adopt new methods; second, to an unwillingness to accept the principle, laid down in the national irrigation act itself, that water should be appurtenant to the land; third, to a fear of jeopardizing capital invested in so-called "floating water rights," which, it was feared, would become valueless if water was attached to the land; fourth, to unwillingness on the part of owners of large holdings to subdivide, as

required by said act, who yet wanted to become beneficiaries under it; fifth, to fear of a merger of the canal systems into one association, and consequent loss of local control; sixth, to ignorance of the plan proposed and the benefits expected; seventh, to pure selfishness and personal interests. One large landowner contended persistently for what he called a "regulation right" (for which he was willing to pay in the same proportion as other landowners), claiming that if the flood waters which belonged to him could be held back in the Government reservoir and the flow "regulated," he would have all the water needed for his 7,000 acres. To have conceded this point would have been equivalent to a complete evasion of the provisions of the reclamation act, limiting the acreage on which a water right could be issued to 160 acres.

Naturally there was much discussion over the objects of the association; the power to be given to a centralized administration; the size of the reservoir district; the amount of capital stock, by whom it should be owned, how acquired, how canceled, and the rights incident thereto; the appurtenancy of water to land; the form of organization; the revenues, how raised; the expenditures, how safeguarded, etc.

The articles of incorporation, completed early in February, 1903, having been approved and adopted by the majority of the members of the executive committee, were submitted to the water-storage conference committee and by them approved and adopted, each one separately and then as a whole.

At once the articles were filed in the county recorder's office and with the secretary of the Territory of Arizona, and duly published as required by law, and then the Salt River Valley Water Users' Association became a legal entity.

The next step was to provide for the creation of a lien on the land, so that the Government might be assured of ample security for the cost of the construction of the reservoir, as required under the law. Without delay the articles were reprinted in contract form for signature by those who thereby became members of the association, and the work of procuring subscriptions to these formal contracts of membership was begun under the direction of a subscription committee, who divided the valley into districts and appointed in each district the best men to be had as solicitors, with instructions to see every landowner and urge early action in "signing up." These contracts called for the signature of the landowner, and wife if married, and a legal acknowledgment by each signer before a notary public.

The shares, when so subscribed for, became appurtenant to the land, one share, and no more, being allotted to each acre of land, under the articles of association, which also limited the voting right of any one landowner to 160 shares.

From time to time, in order to hold up and stimulate the general interest, public meetings were held in various sections of the valley.

Throughout the entire campaign the local press without exception did valiant work, not only in disseminating news of the progress made from day to day, but also by pointed and vigorous editorials clearly setting forth the situation, unqualifiedly indorsing the movement, and urging all landowners to "sign up" and become members of the association without unnecessary delay.

This work was continued with varying success, but persistently, hopefully, and without cessation from early in February to July 17, 1903, when the books were closed with a total of about 200,000 acres subscribed.

A report was then made to the Government, embodying a brief; a complete list of subscribers, giving date of subscription, names, and acreage; a map in colors, showing at a glance what lands and how much each subscriber has signed up, and the grand total; a copy of the articles of association; and also a copy of the by-laws, so far as drawn (it being the policy of the association to draft only such by-laws as experience from time to time showed to be wise and needed). These having met with the general approval of the Secretary of the Interior, the preliminary work of construction was begun by the reclamation service in the fall of 1903, and the first large contract, for the construction of the power canal, was signed by the Secretary of the Interior in March, 1904, and thereafter the work was pushed energetically toward completion.

REPORT OF THE EXECUTIVE COMMITTEE OF THE WATER-STORAGE CONFERENCE COMMITTEE OF THE SALT RIVER VALLEY.

PHOENIX, ARIZ., *October 2, 1902.*

The executive committee of the water-storage conference committee of the Salt River Valley respectfully submits to the general committee the following report:

We have given the most careful, exhaustive, deliberate, and impartial consideration to the problem of the formulation of a comprehensive general plan which would remove the obstacles to be overcome, and enable the landowners and water users of the Salt River Valley to unite their interests and influence in a combined and determined effort to secure the construction of the Salt River reservoir by the National Government.

We have unanimously agreed upon the general plan hereinafter set forth as one which will, in our judgment, if it is adopted and carried into effect by the people of the Salt River Valley, enable them to obtain for themselves the great benefits which water storage would bring to this community and to every member of it.

We believe success in this movement depends wholly upon the people themselves. With a united community there is every assurance of success, and we urge upon every citizen of the Salt River Valley the imperative and urgent importance in this critical period in the history of our development of working shoulder to shoulder to carry out the plan suggested by this committee and set forth in this report.

The national irrigation act does not contemplate or authorize the construction of a reservoir by the National Government for the storage of water to be delivered to a canal company, and by that canal company carried and delivered to the water users, where the canal system is operated and controlled by neither the Government nor the water users. The water must be delivered and distributed to the land to which the right to its use has by the Government been made perpetually appurtenant, either through a Government canal or through a canal owned, operated, and controlled by the owners of the lands entitled to the water.

It is for this reason necessary that such of the canal companies or systems in the Salt River Valley as will carry water from the reservoir to the lands on which the Government will locate reservoir rights shall be transformed into cooperative landowners' companies or merged into one such company, which shall undertake the duty of distributing the water from the reservoir so that every landowner holding a Government reservoir right shall be assured of a corresponding right to have his water carried to him from the reservoir to the land on which he may be entitled to use the water for irrigation. The storage of the water is not the only thing to be considered. Its distribution under a plan which will conform to the requirements and provisions of the national irrigation act is equally necessary and important, and must be arranged and provided for before we can secure the construction of the reservoir by the Government.

It is likewise necessary, in view of the existing local conditions in the Salt River Valley, that some plan should be adopted whereby the Government can deal with all the owners of lands to which reservoir rights are to be issued as a unit.

The Government would not and could not undertake to collect water rates from a large number of landowners for water delivered to them from the reservoir through canal systems over which the Government had no control, and we believe that no plan could now be devised or carried into effect under which the Government would assume control or acquire the ownership of the distributing systems in the Salt River Valley. It must be recognized that this is impossible.

A plan must be adopted which will guarantee and insure to the Government the prompt payment in full when due of each and every annual installment of the total cost of the reservoir in each of the years during which these installments will be due and payable to the Government. The Government must be assured that these payments will be made beyond any question, and that the Government will never be involved in any complications or entanglements. As to these matters, the position of the Government must be made absolutely safe and certain, and we can not expect it to act until that has been done.

We therefore recommend, as suggested in the plan proposed to the committee by Judge Joseph H. Kibbey, that we should first organize the association of water users contemplated by the national irrigation act itself. The shares in this association should be perpetually and inseparably appurtenant to the land on which they are originally issued.

This association, when formed, should embrace all landowners desiring to acquire rights to water from the reservoir, and should be the agency to receive the water from the Government and distribute it to the landowners entitled to its use, or to the respective canals controlled and operated by such landowners, to be by such canals carried to the lands to be irrigated, under such rules and regulations as may be established by the association of water users and the respective canal systems, and approved by the Secretary of the Interior.

This association of water users should also be the agency through which all collections from landowners of moneys due the Government should be made and through which the full amount of each annual installment due the Government each year should be paid all at one time in a single lump sum each and every year when the same is due.

It should be borne in mind that the national irrigation act contains the following clause:

"That nothing in this act shall be construed as affecting or intended to affect or to in any way interfere with the laws of any State or Territory relating to the control, appropriation, use, or distribution of water used in irrigation, or any vested right acquired thereunder, and the Secretary of the Interior, in carrying out the provisions of this act, shall proceed in conformity with such laws."

The foundation of the whole structure upon which the national reservoir and all rights to water from it and every detail of the plan of distribution of that water is that under no circumstances that can arise shall there ever be any interference in any way with any vested right to water from the Salt or Verde rivers. All such rights will be protected by the laws of the Territory and of the National Government, and the plan we propose does not contemplate any surrender or modification of any of those rights. They will continue to exist just as though no reservoir was contemplated or built. We think, however, that it would be desirable that all such rights should be adjudicated at as early a date as may be feasible or practicable. It is not necessary, however, in our judgment, that this should be done as a condition precedent to the construction of the reservoir or before its construction should be authorized and begun by the Government.

It must be further borne in mind, and the fact should never be lost sight of, that the national irrigation act further provides that—

"No right to the use of water for land in private ownership shall be sold for a tract exceeding 160 acres to any one landowner, and no such sale shall be made to any landowner unless he be an actual bona fide resident on such land or occupant thereof residing in the neighborhood of such land, and no such right shall permanently attach until all payments therefor are made."

No landowner will be able to acquire a grant of a reservoir right from the Government unless he shall, before the completion of the reservoir, have brought himself clearly within the letter and spirit of the act in these particulars, and this will necessitate the subdivision, before the completion of the reservoir, of much of the land in the valley into 160-acre tracts where now held in larger tracts.

After a full consideration of the matter we have decided that it will be for the best interests of the people of the Salt River Valley to recommend the construction of the reservoir with a capacity of 1,478,750 acre-feet, the approximate estimate of the cost of the same being $2,549,231.

With a view of carrying into effect the foregoing general plan the following resolutions were adopted by your committee:

Resolved, That it is the sense of this executive committee that a district should be created within Maricopa County, Ariz., composed of land under the present canals now distributing water within the following-described boundaries:

Commencing at a point on the right bank of Salt River at the mouth of the Verde River; thence in a southerly direction to the headgates of the Arizona canal; thence in a general westerly direction along the north line of the right of way of the Arizona canal to the end thereof, at or near the quarter corner between sections 5 and 6, township 3 north, range 1 east; thence west to the left bank of the Agua Fria River; thence in a southerly direction along the left bank of the Agua Fria River to a point in section 14, township 1 north, range 1 west, where the line of the St. Johns canal intersects the Agua Fria River; thence in a southeasterly direction along the north line of the right of way of the St. Johns canal to the north bank of the Salt River; thence across the Salt River to a point in section 35, township 1 north, range 1 east, where the east line of the Gila River Indian Reservation intersects the south bank of the Salt River; thence in a southerly direction along the east line of the said Gila River Indian Reservation to the township

line between townships 1 and 2 south; thence following the boundary of the said Gila River Indian Reservation east along the said township line to the range line between ranges 4 and 5 east; thence south between ranges 4 and 5 east; thence east, between townships 2 and 3 south, to the east line of the right of way of the east branch of the Consolidated canal; thence north, following the east line of the right of way of the east branch of the Consolidated canal to the south line of section 2, township 2 south, range 5 east; thence east to the section corner common to sections 5, 6, 7, and 8, township 2 south, range 6 east; thence north between sections 5 and 6 east, township 2 south, range 6 east, to the east bank of the Highland canal; thence in a generally northern direction along the east bank of said Highland canal to the left bank of the Salt River; thence in a generally northerly direction along the left bank of Salt River to a point opposite the mouth of the Verde River, and thence across Salt River to the place of beginning.

It is further

Resolved, That a water-users' association should now be organized, and that subscriptions should be opened for shares of stock therein, each landowner within the above district to be entitled to subscribe for one share of stock for each acre of his land, each subscription for such share of stock to embody an application for and an agreement to take a reservoir right for the same acreage, or such portion thereof, as may be authorized by the Secretary of the Interior.

It is further

Resolved, That it is the sense of this committee that the lands now under cultivation should be given the first opportunity to secure reservoir rights, and that reservoir rights should be allotted only to holders of shares in the said water-users' association, and this committee believes that under a system of water storage, in view of the water-supply records of the Verde and Salt rivers as prepared by the United States Geological Survey, and the court water commissioner of Maricopa County, that it would be unwise to attempt to cultivate more than 200,000 acres with the entire known available water supply—this supply including the natural appropriated flow of the Salt and Verde rivers, the flood waters to be stored, and the water to be pumped by the power developed at the Salt River reservoir, it being estimated that the amount so pumped will approximate a supply of not less than sufficient to irrigate 20,000 acres.

It is further

Resolved, That the foregoing resolutions be presented to the Secretary of the Interior as the earnest recommendation of this committee.

It must be understood that all the recommendations contained in this report and embodied in the plan proposed are subject to any modification which should be advised by the Secretary of the Interior under the authority given him by the national irrigation act, and that we can do no more than recommend to him what in our judgment is the best plan to be adopted.

It was further

Resolved, That the number of shares of the capital stock of the water-users' association should be limited to 250,000 shares, and that the flat price per acre, which would be the par value of said shares, be fixed at $12.50 per acre, payable in ten equal annual installments of $1.25 per acre per year each, commencing at whatever time shall be fixed by the Secretary of the Interior as the time when the first installment is due the Government upon the reservoir rights to be issued by it.

It was further

Resolved, That a subscription book be prepared containing the report of this executive committee, followed by a statement that the undersigned approve the foregoing plan, and will subscribe for the number of shares and acreage set opposite their respective names, subject to their approval of the articles of incorporation and by-laws of the merger association to be hereafter prepared, and that the said book be pre-

sented to the general committee at its meeting on Thursday, October 2, 1902, and that if the said report of this committee be adopted by the general committee the subscription list should be immediately opened, and that the members of the executive committee and general committee, and any other persons desiring to do so should have the privilege of signing the same.

ARTICLES OF INCORPORATION.

Know all men by these articles of incorporation that we, the undersigned, have associated ourselves together under the laws of the Territory of Arizona as a body corporate.

ARTICLE I.

The name of the corporation shall be and is Salt River Valley Water Users' Association.

ARTICLE II.

The names of the incorporators are: Frank T. Alkire, E. J. Bennitt, J. Wilfred Broomell, A. J. Chandler, Vernon L. Clark, O. H. Christy, William Christy, W. W. Dobson, W. S. Dorman, B. A. Fickas, B. A. Fowler, S. S. Green, C. R. Hakes, James G. Hammels, C. T. Hirst, H. A. Hughes, C. C. Hurley, P. T. Hurley, James Johnson, Emery Kays, W. J. Kingsbury, Wallace A. MacDonald, H. C. Mann, M. W. Messinger, Lin B. Orme, F. H. Parker, Thomas W. Pemberton, Alfred J. Peters, J. E. Price, L. J. Rice, L. D. Rousseau, H. Simkins, M. A. Stanford, C. S. Steward, J. A. Stewart, S. S. Stout, J. E. Sturgeon, Fred Tait, Walter Talbot, G. E. Tompkins, Howard L. Underhill, H. G. Van Fossen, A. P. Walbridge, W. H. Wallace, Henry E. Ware, J. C. Wasson, Sam F. Webb, E. W. Wilbur, W. A. Wilson, but others may become members of said association by subscribing to these articles of incorporation, or a copy thereof, or by the transfer of stock to them in the regular course of the administration of the affairs of the association.

ARTICLE III.

The principal place of transacting the business of the association shall be at the city of Phoenix, in the county of Maricopa, in the Territory of Arizona.

ARTICLE IV.

SECTION 1. The objects for which the association is organized and the general nature of the business to be transacted by it shall be and are:

To provide for and distribute and furnish to the lands of the holders of shares of said association to which said shares and the rights and interests represented thereby are appurtenant an adequate supply of water for the irrigation of said lands.

To divert water within the Territory of Arizona from the public sources of water supply, to impound water and develop sources of water, to pump water from underground sources, to carry and distribute water for the irrigation of the lands aforesaid, and to construct, purchase, lease, condemn, or otherwise in any way whatsoever acquire and own, hold, have, use, control, maintain, preserve, manage, operate, and conduct the means therefor and any and all rights, reservoirs, dams, canals, ditches, flumes, head gates, pipes, machinery, wells, pumps, pumping plants, power houses, transmission lines, and property, both real and personal, of every nature and kind whatsoever necessary or appropriate for the accomplishment of any of the objects or purposes aforesaid.

To divert, impound, develop, pump, distribute, deliver, and use water for all beneficial uses and purposes from surface and subterranean sources by any means adapted

to any of the purposes aforesaid, and create, transmit, and use power for the accomplishment of any of the purposes or objects of the association as hereinbefore set forth.

To have and exercise all the powers, and do all and everything necessary, suitable, convenient, or proper for the accomplishment of any of the purposes or the attainment of any one or more of the objects herein enumerated or incidental to the powers herein named or which shall at any time appear conducive or expedient for the protection or benefit of the association or its members as shareholders therein.

SEC. 2. For the accomplishment or to aid in and promote the accomplishment of the aforesaid purposes or objects, or any of them, this association shall have power to enter into any contract or other arrangement or undertake in whatsoever manner and by whatsoever means may be deemed proper or convenient therefor, to secure action by or the aid of the United States Government in the construction, erection, or acquisition of any dams, reservoirs, canals, wells, or other works or property or appurtenances necessary thereunto for the storage, development, diversion, distribution, or delivery of water to the lands of the shareholders of this association, or to aid therein, and to enter into any agreement with said Government which may be approved by the Secretary of the Interior or any duly authorized official or representative of the Interior Department with reference to the collection and payment to the Government of any and all moneys which may be due the Government under or by reason of rights issued by the Government to members of this association for the use of water from any reservoir or irrigation works acquired or constructed by said Government, or in the acquisition or construction of which it may have aided, and to comply with any conditions, rules, or regulations prescribed by Congress or by any Executive Department or official of said Government lawfully authorized thereunto, concerning the storage, diversion, delivery, application, or use of any water so stored, developed, or delivered to the shareholders of this association from or by means of any works constructed or acquired by the Government or in the construction or acquisition of which the Government may have aided, or which it may hereafter construct or acquire, or in the construction or acquisition of which it may hereafter aid.

SEC. 3. The territory within which are the lands to be irrigated as aforesaid shall be known as the Salt River Reservoir District, and shall include lands within the boundaries described as follows, that is to say: Commencing at a point on the right bank of Salt River at the mouth of the Verde River; thence in a southerly direction to the head gates of the Arizona Canal; thence in a general westerly direction along the north line of the right of way of the Arizona Canal to the end thereof, at or near the quarter corner between sections 5 and 6, township 3 north, range 1 east; thence west to the left bank of the Agua Fria River; thence in a southerly direction along the left bank of the Agua Fria River to a point in section 14, township 1 north, range 1 west, where the line of the St. Johns Canal intersects the Agua Fria River; thence in a southeasterly direction along the north line of the right of way of the St. Johns Canal to the north bank of the Salt River; thence across the Salt River to a point in section 35, township 1 north, range 1 east, where the east line of the Gila River Indian Reservation intersects the south bank of the Salt River; thence in a southerly direction along the east line of the said Gila River Indian Reservation to the township line between townships 1 and 2 south; thence following the boundary of the said Gila River Indian Reservation, east along the said township line to the range line between ranges 4 and 5 east; thence south between ranges 4 and 5 east; thence east, between townships 2 and 3 south, to the east line of the right of way of the east branch of the Consolidated Canal; thence north, following the east line of the right of way of the east branch of the Consolidated Canal to the south line of section 2, township 2 south, range 5 east; thence east to the section corner common to sections 5, 6, 7, and 8, township 2 south, range 6 east; thence north between sections 5 and 6 east, township 2 south, range 6 east, to the east bank of the Highland Canal; thence

in a general northerly direction along the east bank of said Highland Canal to the left bank of Salt River; thence in a general northerly direction along the left bank of Salt River to a point opposite the mouth of the Verde River to the place of beginning; together with any public or other lands on which rights to the use of water from reservoir or irrigation works referred to in section 2 of this article shall be issued by the Secretary of the Interior.

ARTICLE V.

SECTION 1. The capital stock of the association shall be $3,750,000, and be divided into 250,000 shares of the par value of $15 per share.

SEC. 2. Those and those only who are owners of lands, or occupants of public lands having initiated a right to acquire the same, within the territory described in Article IV of these articles of incorporation, or within such extensions thereof as may be hereafter made from time to time under the powers herein conferred for that purpose, shall be the holders or owners of shares of the capital stock of this association. For each acre of such lands shareholders may become the owner of one share of stock of this association and no more.

SEC. 3. As a condition of continued ownership of said shares of stock, and participation in any of the benefits thereof, each subscriber therefor, or transferee thereof in case of transfer, shall, as soon as the right or rights hereinafter referred to become subject to application and acquisition under the rules and regulations prescribed or to be prescribed for the purpose by Congress or any Executive Department of the Government, apply for, and in good faith comply promptly with all such rules and regulations for the acquisition of, a right to the use of water from any source of supply provided by the Government, or in the provision of which the Government has aided, for the irrigation of the lands to which said shares and rights represented thereby are appurtenant. Upon the failure of the subscriber, or holder otherwise, of any of the shares of the capital stock of this association to apply for such rights, or, having applied therefor, upon failure to promptly and in good faith comply with all rules and regulations prescribed or that may be prescribed by Congress or by any Executive Department of the Government relative thereto, then he shall forfeit to the association such shares of stock and all and every right in anywise theretofore or then incident thereto, or that might in anywise arise or accrue from or exist by virtue of such subscription or ownership, or that could by any means arise or be claimed therefrom, and such person, his heirs and assigns, shall thereafter have no right whatsoever as a member of this association by virtue thereof.

SEC. 4. Any shares of stock so forfeited shall at once be canceled and shall not again, under any circumstances, be renewed, revived, or reissued. Other stock in lieu thereof up to the limit of the total number of shares authorized by these articles may be subscribed for and issued, subject to all the conditions of these articles and to the approval of the Secretary of the Interior.

SEC. 5. The ownership of each share of stock of this association shall carry, as incident thereto, a right to have delivered to the owner thereof water, by the association, for the irrigation of the lands to which such share is appurtenant.

SEC. 6. The amount of water so to be delivered to such owner shall be that proportionate part of all stored and developed water, the storage or development of which is or may be effected by this association, or by means of works under its control, management, or direction, or which may become available for distribution by this association from irrigation works built by the National Government, during any irrigating season, as the number of shares owned by him shall bear to the whole number of valid and subsisting shares of the association issued and then outstanding, to be delivered to and upon said lands at such times during such season as he may direct.

SEC. 7. And there shall also be incident to such ownership of such shares the right to have delivered to the owner thereof, for the irrigation of said lands, as the association shall from time to time acquire means for that purpose, the water heretofore, and before the shareholder or his transferee became a member of this association, appropriated by him or by his predecessors in interest, for the irrigation of said lands: Provided, however, that the whole amount of water actually delivered from all sources shall not exceed the amount necessary for the proper cultivation of said lands.

SEC. 8. The records of the association and each and every certificate or other evidence of ownership of shares of stock in the association, when issued, shall contain a description of the lands to be irrigated, and to which the aforesaid rights and shares shall be perpetually appurtenant, and thereafter all rights, whatever their source or whatever their manner of acquisition, to the use of water for the irrigation of said lands, shall forever be inseparably appurtenant to said lands, together with the said shares of stock, and all rights and interests represented thereby or existing or accruing by reason thereof, unless such rights shall become forfeited under the provisions of these articles of incorporation, or of by-laws adopted in pursuance thereof, or by operation of law, or by the voluntary abandonment thereof by deed, grant, or other instrument, or by nonuser for the term prescribed by law; but no such abandonment shall be for the benefit of any person designated by such shareholder, directly or indirectly, or to his use, nor confer any right whatsoever upon the holder of any grant, release, waiver, or declaration of abandonment of whatever kind of such right: Provided, however, that if for any reason it should at any time become impracticable to beneficially use water for the irrigation of the land to which the right to the use of the same is appurtenant on the said land, the said right may be severed from said land and simultaneously transferred and attached to other lands to which shares of stock in this association are or are thereby made appurtenant, if a petition for leave to make such transfer and showing the necessity therefor shall have first been approved by a two-thirds vote of the board of governors at a regular meeting and by the Secretary of the Interior; and all the provisions and agreements hereinbefore in this section contained shall be set forth in the aforesaid certificate or other evidence of the ownership of shares of stock in the association, and such certificate or other instrument shall be signed, executed, and acknowledged by the president and secretary of the association, and by the person to whom it is issued in the manner required by law for the execution and acknowledgment of deeds for the conveyance of real estate, and the council shall pass by-laws prescribing the form of such certificate or other instrument not inconsistent with these articles.

SEC. 9. Every transfer of the title to any lands to which the said rights and stock are so appurtenant, whether by grant or by operation of law (except where the land may be subjected by grant or involuntarily under any law to an easement, the exercise of which does not interfere with the cultivation of the soil by the servient owner) shall operate, whether it be so expressed in the grant or other means of transfer or not, as a transfer of all rights to the use of water for the irrigation of said lands, and all rights arising from, or incident to, the ownership of such stock, and as well as the stock itself, to the grantee or successor in title of said lands; and upon presentation to this association of proof of any such transfer of land, to which such rights are appurtenant, the proper officer shall transfer such stock upon its books to the successor in title to said lands.

SEC. 10. Any transfer, or attempted transfer, of any of the shares of stock of this association, made or suffered by the owner thereof, unless simultaneously a transfer of the land to which it is appurtenant is made or suffered, shall be of no force or effect whatsoever for any purpose, and shall confer no rights of any kind whatsoever on the person or persons to whom such transfer may have been attempted to be made.

SEC. 11. No payments for the capital stock of this association shall be required except in the manner following:

Whenever it shall be announced or otherwise made known by the Government, or any of its proper agencies, to the subscribers to the shares of said capital stock that rights may be initiated to the use of water from any works acquired or constructed by it for the storage or development of water capable of being used on the lands of the subscribers for said shares of stock, or in the acquisition or construction of which works the Government shall have aided, then the said subscribers shall, under the rules and regulations prescribed by the Government therefor, within a reasonable time, apply to the proper agency for such rights at the rate of one acre for each share of stock so subscribed for, and upon proper proof to this association that such rights have been allotted to the subscriber and that he has complied with the Government rules and regulations up to that time, so as to entitle him to complete his ultimate acquisition thereof, then such subscriber shall be deemed to have paid on his subscription of said stock the amount that he shall have then paid to or for the use of the Government for such right, and when all subsequent payments required by the Government for such rights shall have been paid by him, or by anyone for him, then such stock shall be deemed and held to have been fully paid up.

SEC. 12. If it should be determined by the Government that the amount of water that may safely be estimated to be capable of being stored, or developed by works acquired or constructed or to be acquired or constructed by it, or by this association, in addition to the amount of water now appropriated out of Salt River and Verde River, for the irrigation of lands in said reservoir district, shall together be insufficient to properly irrigate 250,000 acres of land, then the number of shares of the capital stock of this association shall be reduced so that the number of such shares shall not exceed the number of acres estimated by the Government to be capable of irrigation from such combined sources of supply.

SEC. 13. If at the time of the determination by the Government of the number of acres capable of such irrigation there shall have been subscribed for a number of the shares equal to the number of acres so estimated to be capable of irrigation, then no subscription for more shares shall be taken. If the number so subscribed for, however, shall then exceed the estimated number of acres so capable of irrigation, then there shall be allotted to said subscribers that number of shares equal to the estimated number of acres capable of irrigation. In such allotment cultivated lands shall have the preference; and any excess in the number of shares subscribed for over the number so allotted shall be canceled, and thereafter shall not be issued.

SEC. 14. In exercising the right of preference to allotment of shares, provided for in the foregoing section, the by-laws may prescribe reasonable times within which such preference shall be declared and the manner thereof.

ARTICLE VI.

SECTION 1. The exercise of the corporate powers of this association and the management of its affairs shall be vested in—

(1) A council;

(2) A board of governors;

(3) One or more local boards of water commissioners, and

(4) A president, vice-president, treasurer, secretary, and such other officers and agents as shall or may be, from time to time, created and established by the council.

SEC. 2. The council shall consist of 30 members, who shall hold their offices for three years. In the event of a vacancy occurring from death, resignation, ceasing to be a shareholder of this association, removal from the reservoir district, or other cause, the unexpired term shall be filled at the annual election next after the vacancy shall have occurred. Shareholders of this association only shall be eligible to the office of member of the council.

SEC. 3. The annual election of the members of the council and of the other officers for whose election these articles provide shall be held on the first Tuesday of April in each year.

SEC. 4. For the purpose of electing members of the council the territory described in Article IV of these articles shall be divided by the council into ten districts, which districts shall severally consist of contiguous territory, and be as nearly uniform in shape, and contain as nearly an equal number of acres to which shares of stock in this association are appurtenant, as may be practicable.

SEC. 5. At each annual election, after 1904, there shall be elected one member of the council from each of said several districts by the electors thereof. Such member shall at the time of his election be the owner of lands situated within the district for which he is elected to which shares of stock of this association are appurtenant, and shall also be a resident of the reservoir district, and if he should, during his term of office, cease to be such owner, or a resident of such reservoir district, his office shall thereupon, and by reason thereof, become vacant.

SEC. 6. At the annual election to be held on the first Tuesday of April, 1904, there shall be elected three members of the council from each of said districts, one of whom shall be elected to serve for one, one for two, and the third for three years.

SEC. 7. The term of office of members of the council shall begin on the first Monday in May following their election.

SEC. 8. Until the election of members of the council at the annual election in 1904, and until their qualification, the council shall consist of the following-named persons, that is to say: J. Wilfred Broomell, Vernon L. Clark, O. H. Christy, W. W. Dobson, B. A. Fickas, S. S. Green, C. R. Hakes, C. T. Hirst, P. T. Hurley, James Johnson, Emery Kays, W. J. Kingsbury, Wallace A. MacDonald, H. C. Mann, M. W. Messinger, Thomas W. Pemberton, Alfred J. Peters, J. E. Price, L. J. Rice, L. D. Rousseau, C. S. Steward, J. A. Stewart, J. E. Sturgeon, Fred Tait, G. E. Tompkins, Howard L. Underhill, H. G. Van Fossen, A. P. Walbridge, Henry E. Ware, W. A. Wilson.

SEC. 9. The council shall meet at least once in each year at the city of Phoenix, in Maricopa County, Ariz. This regular annual meeting of the council shall begin on the first Monday in May in each year and continue in session at its pleasure.

SEC. 10. Special meetings may be called and held in such manner and at such times and under such provisions as may be prescribed by the by-laws. The general object and nature of business to be transacted at any special meeting shall be made known by reasonable public notice, such notice to be that prescribed by the by-laws therefor.

SEC. 11. The members of the council shall serve as such without compensation, but may receive mileage one way at the rate of 10 cents per mile for each day of actual attendance.

SEC. 12. No member of the council shall be eligible to any office of this association to which there is attached any emolument or compensation, nor shall he be so eligible until at least two years shall have elapsed after his term of office as councilman shall have expired; provided, that this inhibition shall not apply to any person by reason of his membership of the first council.

SEC. 13. The council shall have power to enact and adopt and provide for the enforcement thereof by-laws for the government of the members of this association and the management of its business, and the conduct of its affairs, and to repeal, modify, and amend the same from time to time. But the council shall not have power to adopt or to enforce any by-laws that in anywise conflict with any rule or regulation established by the Secretary of the Interior or other agency of the Government for the administration of water from any reservoir or other works acquired or constructed by the National Government, or in the acquisition or construction of which it shall have aided, and which may be used for supplying water to the lands of the shareholders of this association.

SEC. 14. No by-laws shall be passed or enforced which shall interfere with or affect any present existing vested right of any member of this association to the use of water for irrigation.

SEC. 15. All by-laws shall be of general application so far as general laws can be made to apply.

ARTICLE VII.

SECTION 1. At all elections the electors shall possess the following qualifications:

(1) Shall be at the time of the election the owner of at least one share of the capital stock of this association, and shall have been the owner thereof, as shown by the books of the association, for at least twenty days before such election.

(2) Shall be of the age of 21 years or more and of sound mind.

SEC. 2. At all elections each shareholder shall be entitled to one vote for each share of stock owned by him, not, however, to exceed in the aggregate 160 votes, and no more.

SEC. 3. The votes shall be by written or printed ballot, and be voted only by the electors at the polls in person.

SEC. 4. The council may make reasonable by-laws for the registration of voters and the method of holding elections.

SEC. 5. At all elections the person receiving the highest number of votes for any office shall be deemed elected to such office.

ARTICLE VIII.

SECTION 1. The board of governors shall consist of the president, by virtue of his office, and ten other members. Each of said ten members shall be the owner of lands situated within the district for which he is elected, to which shares of stock of this association are appurtenant, and shall also be a resident of the reservoir district; and if he should, during his term of office, cease to be such owner or a resident of such reservoir district, his office shall thereupon, and by reason thereof, become vacant.

SEC. 2. One member of said board shall be elected at the annual election to be held on the first Tuesday of April, 1904, and at each annual election thereafter from and by the electors of each of the districts into which the territory described in Article IV of these articles is or may be divided for the purpose of the election of members of the council.

SEC. 3. Until the election of members of the board of governors at the annual election in 1904, and until their qualification, the board of governors shall consist of the following-named persons, that is to say: Frank T. Alkire, A. J. Chandler, William Christy, B. A. Fowler, Lin B. Orme, F. H. Parker, H. Simkins, M. A. Stanford, W. H. Wallace, Sam F. Webb, E. W. Wilbur.

SEC. 4. If a member of the board of governors at any time during his term of office should cease to have any of the qualifications prescribed for that office, such office shall thereupon become vacant, and in the event of a vacancy from that cause or by reason of the death, removal from the reservoir district, or resignation of any member of said board of governors, the vacancy shall be filled by the council if it occurs more than sixty days prior to a regular election.

SEC. 5. The board of governors shall meet in regular session at the office of the association on the first Monday of each month, unless it be a legal holiday, in which case the board shall meet on the following day.

SEC. 6. Special meetings of the board of governors may be called by the president, or by any six of the members of the board. Such call shall be in writing and signed by either the president or any six of the members, and shall state the time of such proposed meeting and the nature of the business to be transacted thereat. Such written call shall be filed with the secretary, who shall thereupon immediately, and

at least five days before the time fixed for such meeting, mail, postpaid, to the president and each member of the board a copy of such call, and shall publish the same in some newspaper published and of general circulation in the territory described in Article IV of these articles on three consecutive days before and exclusive of the day fixed for such special meeting. If the secretary fail or refuse to publish such call or to mail copies thereof, as above provided, then either the president, if he issued the call, or any one of the members who issued the same, may make publication and mail copies of the call, with like effect as if done by the secretary. Special meetings of the board of governors shall be held at the office of the association.

SEC. 7. The members of the board of governors shall receive such compensation as shall be prescribed by the council by by-laws.

SEC. 8. The board of governors shall have the administration of the corporate affairs and business of the association, and shall manage and conduct the same subject to all the provisions of these articles and of the by-laws.

SEC. 9. It shall have the power to appoint, subject to removal by it at any time, a general superintendent, and prescribe his duties and powers, subject to all rules and regulations prescribed by the by-laws, and to employ an engineer or engineers and such other employees as may be proper and necessary to effect the purposes of this association, subject to such rules and regulations as may be prescribed by the by-laws.

SEC. 10. It shall have the power to prosecute, defend, and compromise all lawsuits; to make all contracts, in the name of the association, necessary and proper for the conduct of the affairs and the carrying on of the business of the association, subject to all limitations and regulations prescribed by these articles or the by-laws.

SEC. 11. It shall have the power to estimate, make, and levy all assessments against the shareholders of this association to the extent and in the manner authorized by these articles and regulated and prescribed by the by-laws.

SEC. 12. It shall have the power to make, publish, and enforce rules and regulations concerning the distribution, use, and application of water for irrigation, subject at all times to and not inconsistent with these articles or with the by-laws or with any rules or regulations established by the Secretary of the Interior or other agency of the National Government.

SEC. 13. It shall keep, or cause to be kept, a record of its transactions, which shall at all times remain in the office of the association, and shall, during office hours, be open to the inspection of the shareholders or their properly authorized agents.

SEC. 14. It shall hear and determine complaints of shareholders of nonservice or of improper service or distribution of water or of improper performance of duty by any employee of the association relative to the distribution of water.

ARTICLE IX.

SECTION 1. The lands under each separate distributing canal system within the territory described in Article IV of these articles shall on the petition of the owners of two-thirds of the shares of the capital stock of this association appurtenant to the lands in such proposed canal division be formed into a canal division, and the boundaries of such division be fixed by the council. On the petition of the holders of a majority of the shares in this association owning the lands affected thereby, or without such petition, whenever in the judgment of the council it will be beneficial, the council may change or modify the boundaries of any canal division.

SEC. 2. A board of water commissioners, to consist of three members, shall be appointed annually by the board of governors for each canal division. Such appointment shall be made in the month of May in each year, and no one shall be eligible for such office unless he be a resident within the canal division for which he may be appointed, and a qualified elector as provided in these articles. The term of

office of the members of the boards of water commissioners shall be from the time of their appointment until the first day of May in the following year, and until their successors shall have been appointed and have qualified, and they may be removed for cause by the board of governors, who may fill any vacancy in any board of water commissioners for the unexpired term.

SEC. 3. If a petition signed by shareholders in this association, constituting a majority of qualified electors, residing in any canal division, shall be presented to the board of governors at their first meeting, in May, naming three persons eligible for appointment as water commissioners, in said canal division, and asking for their appointment to such office, such persons shall be appointed as such water commissioners.

SEC. 4. In each canal division the board of water commissioners thereof shall have the control and management of the local affairs of the canal system therein, and of the distribution of water therefrom, subject to the provisions of these articles, and to the by-laws, and to the rules and regulations established by the council, or by the Secretary of the Interior, or any agency of the National Government.

SEC. 5. The board of water commissioners in each canal division may appoint a secretary of the board and employ a division superintendent, removable at their pleasure.

SEC. 6. The board of water commissioners may call a meeting at any time of all the landowners in their canal division who are shareholders in this association, to consider and determine whether an assessment or assessments for their special benefit should be levied on the shareholders in said division. At least ten days' notice of such meeting shall be given by the secretary of said board, by posting a notice thereof in three public places in said canal division, and mailing a copy of such notice, with postage prepaid, to each shareholder residing in said canal division.

SEC. 7. If a majority of such shareholders who are the holders of a majority of the shares of this association appurtenant to lands in said canal division shall at said meeting approve the levy of such special assessment or assessments and sign a petition to the board of governors that the same be levied on the shareholders in this association, owning lands in said division, and specifying the amount of said proposed assessment and the purpose for which it should be levied, and said board of water commissioners should recommend it in writing, indorsed on said petition, it shall be the duty of the board of governors to levy said special assessment on all the shareholders in this association owning lands in said canal division.

SEC. 8. When so levied the said assessment may when collected be disbursed by the board of water commissioners of said canal division for the purpose specified in its levy, and shall be paid out by the treasurer on warrants drawn on him by the said board of water commissioners; provided, however, that nothing in this article shall limit or abridge the right of the association to make, levy, and collect assessments, as elsewhere in these articles provided.

ARTICLE X.

SECTION 1. A president and a vice-president of the association shall be elected at the annual election to be held on the first Tuesday of April, 1904, and every two years thereafter, and shall serve two years from and after the first Monday of May following their election and until their successors are elected and qualified. Until the election of a president and vice-president, as above provided, B. A. Fowler shall be the president and E. W. Wilbur shall be the vice-president of this association.

SEC. 2. A treasurer and a secretary of the association shall be appointed by the board of governors at its first regular meeting in each year, and shall serve for one year thereafter and until their successors have been appointed and have qualified.

SEC. 3. The treasurer and secretary shall give such bond as may be prescribed by

the by-laws; provided that no member of the board of governors nor the secretary shall be accepted as a surety on the bond of the treasurer.

SEC. 4. The president, vice-president, treasurer, and secretary shall perform such duties as are prescribed by these articles and by the by-laws, wherein such by-laws shall not be inconsistent with law or with these articles, and shall receive such compensation as may be fixed by the by-laws, which compensation shall be neither increased nor diminished during their respective terms of office.

SEC. 5. In case of the absence, illness, or inability of the president to act from any cause, or in case of a vacancy in that office, the vice-president shall act in the place and stead of the president. In the event of a vacancy in the offices of both the president and vice-president, the council shall call a special election to fill the unexpired terms for both offices; provided, however, that such vacancies, if occurring within sixty days before the next annual election, shall be filled by appointment by the council.

SEC. 6. The president shall be the chief executive officer of the association, and shall have general supervision over all other officers of the association in the performance of their duties as such, and of the conduct of the business and affairs of the association. He shall preside at all meetings of the board of governors and shall be ex officio a member of that board, and perform such other duties as may be devolved upon him by the by-laws.

SEC. 7. All certificates or other evidences of the ownership of shares of stock in the association issued by the association shall be signed by the president and secretary of the association and shall have the seal of the association affixed thereto.

SEC. 8. All contracts and instruments in writing executed for or in behalf of the association shall be so executed in the name of the association by the president and secretary, and shall have the seal of the association affixed thereto.

SEC. 9. The treasurer shall receive and be the custodian of all moneys and other funds of the association. No moneys shall be paid out by the treasurer unless upon warrants drawn on him by the president and secretary, except as provided in Article IX, section 8, of these articles. No warrants shall be drawn on the treasurer by the president and secretary except upon the order of the board of governors, recorded in the minutes of the board, and in a warrant record to be kept by the secretary.

SEC. 10. The treasurer shall keep a full, complete, and accurate account of all moneys of the association received and disbursed by him, in books belonging to the association, and shall present a report and account thereof for the last preceding month to the board of governors at every regular monthly meeting of said board, and shall, at the expiration of each quarter, prepare and publish, in such manner as the by-laws may prescribe, a quarterly statement to shareholders showing all such receipts and disbursements during the last preceding quarter; and the treasurer shall perform such other duties as may be devolved upon him by the by-laws.

SEC. 11. The secretary shall act as the clerk of the board of governors, and keep a record of all their proceedings. He shall be the custodian of the seal of the association, and of all its books, papers, and records. He shall immediately upon their adoption and certification to him by the council, record in a book of by-laws to be kept by him in his office, all by-laws adopted by the council, and shall keep such book open to the inspection of any member of the association at all times during business hours. And the secretary shall perform such other duties as may be devolved upon him by the by-laws.

ARTICLE XI.

The council may create such other offices as may be necessary for the carrying on of the business and affairs of this association, and prescribe the manner of appointment, powers, duties, terms of office, eligibility, and compensation thereof by by-laws not inconsistent with these articles.

ARTICLE XII.

Upon proper complaint thereof by the board of governors, duly sustained, the council may remove from office the president, vice-president, treasurer, secretary, or the incumbent of any office created by the council, for incompetence, neglect of duty, misappropriation of funds of the association, or for violation of any of the provisions of these articles or of any by-law. The council shall prescribe by by-laws the procedure for such removal.

ARTICLE XIII.

SECTION 1. Revenues necessary for the accomplishment of the purposes of this association shall be raised by an assessment thereof, from time to time as required, upon and against the shareholders.

SEC. 2. The council shall have power to make and enforce necessary by-laws for the making, levying, collecting, and enforcing of such assessments.

SEC. 3. Assessments for the ordinary cost of operation, maintenance, and repair of the works of the association, or of those the maintenance and control of which are, or may be hereafter, lodged in the association, shall be equally assessed against all the shareholders in proportion to the number of shares held by them, respectively.

SEC. 4. Assessments for the purpose of constructing or acquiring, or for the betterment, improvement, renewal, replacement, or preservation of any works, property, or rights of the association, or for the purpose of preserving, or increasing, or more efficiently or economically distributing the water supplies available for distribution by the association, or for the fulfillment of any obligation undertaken by the association in any contract, agreement, or other arrangement with the United States Government, or necessary for the accomplishment or carrying out of any of the purposes of the association, may be equally assessed against all the shareholders in proportion to the number of shares owned by them, respectively.

SEC. 5. Assessments for expenditures for purposes that are of benefit to a part only of the shareholders may be specially assessed in proportion to such benefits against such shareholders, but no expenditure to be provided for or covered by such special assessment shall be made, or obligation to expend the same incurred, except upon the petition of the holders of two-thirds of the shares to be so specially benefited thereby.

SEC. 6. Assessments shall become, from time to time as they are made and levied, and until they are paid or otherwise discharged shall be and remain a lien on the lands of the shareholder against which they are levied and upon the shares of stock appurtenant to said lands and all rights and interests represented by such shares. The manner of fixing the lien and enforcing the same shall be prescribed in the by-laws.

SEC. 7. Except for the ordinary operation, maintenance, and repair, no work shall be undertaken, purchase made, or indebtedness incurred or be authorized during any one year whereof the cost or amount thereof shall exceed $50,000 until it shall have first been ratified by at least two-thirds of the votes cast at an election to be called for that purpose. Special elections may be called and held for such purpose under such by-laws as the council may prescribe not inconsistent with these articles.

ARTICLE XIV.

Nothing in these articles of incorporation, or in the fact of becoming a member of this association, shall be construed as affecting, or intended to affect, or in anyway interfere with the present vested rights of any person to the prior use or delivery of the natural appropriated flow of the waters of the Salt and Verde rivers.

ARTICLE XV.

The individual property of the shareholders shall be exempt from liability for incorporate indebtedness of this association.

ARTICLE XVI.

The incorporate indebtedness shall not exceed two-thirds of the amount of the capital stock.

ARTICLE XVII.

This corporation shall endure for the term of twenty-five years.

ARTICLE XVIII.

These articles of incorporation can only be amended by the shareholders at a regular annual election or at a special election called for that purpose. No proposed amendment shall be submitted to the shareholders until it shall have first received the approval of two-thirds of the members of the council at a regular or duly called session thereof, nor shall any such proposed amendment be so submitted until it shall have been published in full at least once in each week for four consecutive weeks in at least three newspapers published and of general circulation within the territory described in Article IV of these articles, the last of which such publications shall be not less than ten nor more than twenty days before any such election.

ARTICLE XIX.

This association may accept and avail itself of, or subject itself to, the provisions of any law or laws enacted or that may be enacted by Congress or the legislative assembly of the Territory or State, when it becomes a State, of Arizona, relative to corporations which may be applicable to corporations organized for like purposes as this association. Such acceptance or subjection shall be valid when ratified by at least two-thirds of the votes cast at any annual election, or at any special election called for the ratification thereof.

ARTICLE XX.

The seal of this association shall be a figure of two concentric circles, the outer being 2 inches and the inner 1½ inches in diameter. In the space between the two shall be the words "Salt River Valley Water Users' Association," and bearing within the center space an unrolled scroll with the words and figures thereon, "Incorporated, 1903, Arizona."

Witness our hands hereto this 4th day of February, A. D. 1903.

RECLAMATION WORK IN CALIFORNIA.

By Hon. GEORGE C. PARDEE, Governor of California.

I am glad to have this opportunity to tell you that I am pleased to note the intelligent and scientific manner in which your department is taking up the development of irrigation in the arid West, particularly in California, where, of course, my interest centers. I realize that your work will be of the greatest benefit to my State, and hold myself in readiness, and the State of California is ready, now as in the past, to cooperate with you in every way to develop as soon as possible the irrigation water supply of California.

I believe this to be as proper Government work as is that of preserving the navigability of our rivers, and, in fact, that it will produce, at this stage of our national growth, greater results than will the promotion of navigation by the Government. To my mind the national work for irrigation in the West should take precedence over river and harbor improvements, and both should be, for many reasons, Government charges. What California needs, even when her rainfall amounts to 15 or 20 inches per annum, is the power to regulate the distribution of rainfall on the land at such times and in such amounts as will best serve the needs of the crops to be produced, and the only way to do this is, of course, to irrigate. While irrigation is not so absolutely necessary in northern as in southern California, all sections of the State would be greatly benefited by its application. Inasmuch as your department has not yet filed a report of the progress of your work, which is being done in cooperation with the State of California, I can not judge accurately of its full scope, but from the reports that have been made from time to time by your engineers in the field and by the statements filed by the board of examiners I think a vast amount of good must come from the records being gathered by your department.

The measurement of streams and run-off from the different drainage areas being taken by your hydrographers is furnishing a fund of information that will be of very great service when irrigation is developed and water becomes of greater value in all parts of the State. I confess that I am getting eager to see the actual construction work begin in California. A conference with Messrs. Savage and Bien, of your department, after they had reviewed the proposed projects on Colorado River, on King River, and at Clear Lake, gives hope that such construction may be expected in the near future. In our efforts at cooperation, both in the hydrographic and topographic, as well as in the forestry departments, it has been the aim of myself and my administration to leave the United States Government absolutely unhampered, so as to get the very best for the whole State. So, in the reclamation service, whether you gentlemen, in your scientific wisdom, find it best to begin actual construction in the southern, central, or northern part of the State, we hope that the very best place will be selected, and that the wisdom of the President's policy for national irrigation may be put in concrete form at an early date in California.

More and more am I impressed with the vastness of the resources of California and of its limitless possibilities when irrigation and cultivation are brought to that high state which will doubtless be attained. Therefore I regard your work in the Geological Survey as of the greatest importance to California.

WORK IN COLORADO.

By A. L. FELLOWS.

PROJECTS CONSIDERED.

During the last year investigations made with a view to reclaiming arid lands in Colorado have been carried on at four localities. Three of these will be mentioned very briefly, but the fourth will be described in detail.

The first investigation made was in northeastern Colorado, on South Platte River, in the vicinity of Sterling. This project was not favorably considered, and a report has been made adverse to construction on the ground that the water supply of the South Platte is insufficient.

Second, may be mentioned the investigations made in northwestern Colorado, which are still under way. These investigations have been carried on more particularly on Yampa and White rivers. Those on the Yampa have been merely reconnaissance surveys, while those on the White have been made in greater detail. I may say that I am favorably impressed with the possibilities in this locality, but it is too early to make detailed statements concerning these projects.

Third, I may mention the investigation on Grand River near Grand Junction, made with a view to reclaiming arid lands in that locality. It was hoped at first that it might be possible to take out the water of Grand River at an elevation sufficient to irrigate the plains in eastern Utah, but the investigation showed that this plan was not feasible. There seems, however, to be a possibility of developing a large tract of land—about 60,000 acres—in the vicinity of Grand Junction by a lower line. No detailed examination has been made of this low-line project, although the people of Grand Junction are desirous that it shall be investigated.

The project that will be of greatest interest to you, I believe, is that which is generally known as the Uncompahgre Valley project, which contemplates the taking of water from Gunnison River at a point in the Grand Canyon of the Gunnison about 16 miles northeast of the town of Montrose. Gunnison River rises in south-central Colorado and flows in a westerly and northwesterly direction to its junction with the Grand. For about 60 miles it flows through a valley which is usually narrow, but which at times broadens to such an extent that some irrigation is practiced on the bottom lands. For the last 20 miles of this course, however, it runs through what is known as the Black Canyon of the Gunnison, which ends at the mouth of Cimarron River where the Grand Canyon of the Gunnison commences. In this deep and rugged canyon, which is almost inaccessible to man, at a point about 20 miles below its head, it is intended that water shall be taken for the reclamation of the Uncompahgre Valley.

The Uncompahgre Valley is broad and level, although the surface of the ground is rolling in places. The discharge of the river varies from 60 second-feet at low water to 1,000 second-feet at high water, an amount totally inadequate to irrigate successfully the entire valley, which comprises about 175,000 acres, of which 120,000 acres are in private ownership. The best lands of the valley have been settled upon for some time, but, owing to the inadequate water supply, have not been irrigated, the Uncompahgre furnishing at its normal stage only about enough water to irrigate 10,000 acres.

The present project contemplates the diversion of the waters of the Gunnison by means of a tunnel about 6 miles in length, with a cross-section 12 by 12 feet, capable of carrying approximately 1,200 second-feet of water. This amount will be discharged into the Uncompahgre Valley at an elevation sufficiently high to irrigate the entire valley. The people there have formed a water-users' association similar to that organized in the Salt River Valley in Arizona, and have subscribed for about 70,000 acres of land. The amount for which it is necessary to subscribe, as informally stated, is 85,000 acres. The maximum amount to which stock may be assessed is $25 per share. This is a high rate, but the lands of the Uncompahgre Valley can well stand it from their adaptability to fruit raising and the value of its other crops. Good fruit lands are easily worth from $300 to $400 an acre, and in some places where there is a satisfactory water supply, their cost is much greater.

It is expected that the total cost of constructing the tunnel and distributing canals will be about $2,500,000.

DENVER OFFICE.

Owing to the fact that Denver is so admirably located with reference to the work of the reclamation service in the Rocky Mountain region, it was decided in the winter of 1902–3 that a permanent office should be established at that point. Until April 1, 1903, office room was obtained through the courtesy of the State engineer of Colorado, as had been the case for the last four years, and with his cooperation and that of his assistants great expense had been saved. Owing, however, to the fact that the work of the reclamation service was growing so rapidly, and that the quarters furnished by the State engineer, although as commodious as possible, were still too cramped to permit satisfactory results, it was decided that rooms should be rented elsewhere. Accordingly, on April 1, three rooms were rented in the Majestic Building, Denver, and these rooms were occupied by the engineers and assistants of the hydrographic branch until July 1, when it was again found necessary to move, for the same reasons that had prevailed in the former instance. More commodious quarters were then found in

the Chamber of Commerce building, at the corner of Fourteenth and Lawrence streets, and the office of the hydrographic branch was installed at that point, and there it has remained ever since.

These rooms have been supplied with the necessary furniture in the way of drawing tables, desks, and other equipment, and it is expected that this office will be made to some extent the center of the work of the reclamation service in the Rocky Mountain region, particularly in the winter time, in order to avoid such crowding in the headquarters at Washington as otherwise would be necessary.

The work at this point is under the immediate supervision of Mr. A. L. Fellows, district engineer. Messrs. H. A. Storrs, John E. Field, Raymond F. Walter, and M. C. Hinderlider, engineers, and a number of assistants have their headquarters at this point also.

TOPOGRAPHIC WORK IN THE GRAND CANYON OF THE GUNNISON.

By I. W. McConnell, C. E., Resident Engineer.

Under authority of the reclamation act of June 17, 1902, the engineers of the Geological Survey are preparing plans for the study of the problem of furnishing an additional supply of water to the Uncompahgre Valley in Colorado. The valley contains in its cultivable parts about 150,000 acres of highly fertile land. A large portion of this is under cultivation, and in many places splendid orchards and fields give unmistakable evidence of the richness of the soil. The water for the present cultivation comes from Uncompahgre River, which furnishes less than one-tenth the quantity that could be used. Many entries have been entirely abandoned and the land covered by them has been allowed to revert to desert conditions owing to lack of water. The productive capacity of all of the lands that are now under cultivation is curtailed from one-half to one-fourth on account of the shortage in the water supply during the late summer and early fall. To remedy this condition it is proposed to divert water from Gunnison River into the Uncompahgre Valley.

Gunnison River rises in central southern Colorado and flows west-northwest into Grand River at Grand Junction, not far from the Utah border. It is a stream whose volume varies from 600 to 20,000 second-feet or more, the minimum flow coming in the winter season. It is proposed to divert 1,200 second-feet into the Uncompahgre Valley. From Sapinero, Colo., to the State bridge in Delta County, a distance of about 60 miles, the Gunnison flows through a profound canyon, a chasm with almost vertical walls varying in depth from 1,000 feet to nearly 3,000 feet. The width is not great, being in some places only 1,700 feet, so that one standing on the rim has spread before him the great rent in the earth with its frightful crags

and its wonderful spires, all so near that its grotesque details and its awful sublimity make a direct appeal to the eye. The Denver and Rio Grande Railway follows the river in the canyon down to the mouth of the Cimarron. Here the obstacles become too great, and the railroad climbs out of the canyon and into the Uncompahgre Valley by way of Cerro Summit. From Cimarron to the State bridge the Gunnison flows through the deepest part of its canyon, which is difficult of access and is in a country unknown to any except adventurous hunters, range riders, and exploring engineers. In the depths of this canyon the Government engineers propose to locate controlling works for the diversion of the water that is to transform the Uncompahgre Valley. The diversion is to be made by a tunnel about 6 miles long, driven through the hills that separate the Gunnison from the Uncompahgre. To determine the details of topography necessary for the location of dams, head gates, and other controlling works, and to intelligently present the project to the executive officers of the Geological Survey, has been one of the problems confronting the engineers intrusted with the work.

The surveys and explorations now complete bring out the following facts:

First. The bottom of the main canyon is accessible at intervals by trails leading into it through cross canyons or rock slides.

Second. The bottom of the main canyon is covered with débris from the cliffs above, the fragments varying in size from pebbles to enormous bowlders containing 200 cubic yards or more. Bed rock is nowhere visible in the floor of the canyon, but the whole course of the stream is over bowlder beds.

Third. The river is confined at the bottom of the canyon to a channel which is never more than 300 feet wide from vertical wall to vertical wall, and which narrows up in places to a width of less than 50 feet. Usually, where the space between vertical walls is more than 100 feet the stream flows against the rock wall on one side, the other side being occupied by a talus slope. The river may be traversed at low water by working back and forth across the stream from talus slope to talus slope. A boat is useful only for crossing, the stretches of water where a boat may be used being very infrequent and very small.

Fourth. The fall of the river amounts to 800 feet in 4 miles. This fall is somewhat regularly distributed, considering the character of the stream. Vertical plunges of more than 10 or 12 feet are rare, the principal rapid being at Torrence Falls, where a drop of 96 feet is encountered in 1,100 feet of horizontal distance. The impression most strongly made upon the observer is that these falls or rapids are the result of enormous rock-falls from the cliffs above, which block the channel. In the course of time the pools so formed silt up and

the water tumbling over the lower edges of the bowlders produces the fall. In this way the entire bed of the river is formed. If bed rock be anywhere near the surface, it is so thoroughly concealed as to be nowhere visible.

Fifth. It is possible to traverse the stream only at low water. A very small increase in the flow makes crossing impossible, and since the torrent flows first against one cliff and then against the other, any progress along the stream bed is out of the question.

The information now in the hands of the engineers of the Geological Survey has been derived from the following sources:

First. Messrs. John E. Pelton, of Montrose, Colo., and J. A. Curtis, of Delta, Colo., conducted a party of five into the canyon in 1900. This party started from Cimarron and traversed the canyon for about 20 miles. At a point just above the dam site proposed in the present work they abandoned the trip, and sought a trail by which to escape from the canyon. They found a side gorge by which, after a whole day spent in arduous climbing, they reached the rim on the north side. Here they found themselves in an arid region without food or water, as they had found it impossible to carry anything out of the canyon. After a memorable struggle they reached some ranches, where their wants were supplied. This party carried an aneroid barometer, a pocket compass, and a surveyor's chain. Their measurements were only approximate, however, so the work can not be ranked as a survey. They started in with two boats, both of which were lost before the trip was ended. The results of this trip were valuable mainly in demonstrating the fact that the canyon was not totally inaccessible.

Second. In 1901 Mr. A. L. Fellows, now district engineer, U. S. Geological Survey, Denver, Colo., taking a single companion, Mr. William W. Torrence, who had been a member of the Pelton party, made an exploring trip from Cimarron to Red Rock Canyon. This trip, made under great difficulty and involving prodigious labor on the part of the explorers, covered practically all that part of the canyon not traversed by the Denver and Rio Grande Railway.

Third. Mr. Fellows, assisted by Messrs. W. P. Edwards and L. E. Foster, U. S. Geological Survey, with two rodmen, ran a traverse and level line from Beaver Camp to Red Rock Canyon. This was the first accurate survey made in the canyon. It covered about 4 miles. It is a striking comment upon the ability and fidelity of the men who made this survey that all the subsequent checks which have been applied to this line have failed to reveal any inaccuracy. When it is remembered that this line consisted of a closed circuit carried down a gulch dropping 2,000 feet in half a mile, thence down the river and out again over a similar gulch, the circuit being closed at the top, and that twenty-seven days were consumed in the task, some appreciation of the difficulties may be had. The traverse work was done with a

No. 1 C Buff & Berger transit and a 500-foot tape. The leveling was done with a 15-inch and a 20-inch Gurley level, with Philadelphia and New York rods. A folding canoe was used for such boating as was possible. The major portion of the distance, however, was covered on foot by climbing steep slopes, scrambling over enormous bowlders, and wading the icy stream, all the while carrying heavy packs of camp equipment or instruments. During the trip one transit was lost by the overturning of the float on which it was being taken across the stream.

Fourth. During the month of July, 1903, the topography of the canyon above and below the dam site was surveyed by parties working under the immediate supervision of the writer. This work made necessary the projection into the canyon of the main triangulation controlling the valley and the establishment of such bases as were required for the sketching of details. The assistants on the work were Joseph A. Sargent, W. P. Edwards, and L. E. Foster, assistant engineers, U. S. Geological Survey. An ordinary transit, reading to twenty seconds, was used on the triangulation, the contours being taken by plane-table methods.

An inspection of the detail map (p. 166) will show that the river runs through a narrow cut only 40 feet wide at the proposed dam site. For the location of controlling works the topography was wanted both upstream and downstream from this point for a distance of 1,500 feet. Above the dam site talus slopes lie alternately, first on one side of the stream and then on the other, but so cut off from one another that a person standing on one talus slope is completely isolated from any other on the same side of the stream unless he climbs out of the canyon and descends to the other slope from the rim. This is sometimes a task equivalent to a day's work, and is possible at all only in those few instances where a rock slide leads from the rim of the canyon down to the slope. Slopes on the north side of the canyon could be reached from slopes on the south side only by making a detour of 150 miles and climbing into the canyon from the north rim, the stage of the water being such as to make crossing the stream on floats an impossibility.

In this survey it was necessary to make use of four trails leading into the canyon. Trail A leads into the canyon below the dam site from the south rim. It was discovered by Mr. Fellows in 1900. From the slopes at the foot of this trail all the topography below the dam site was taken. Trail B is a trail, discovered on July 4, leading into the canyon from the south rim about 1,000 feet above the dam site. This trail leads on to the main talus slope above the dam site on the south side of the river, and from it most of the topography above the dam site was sketched. Trail C is midway between trails A and B and leads on to an otherwise inaccessible talus slope or gravel bar imme-

diately above the dam site designated as Fosters Slide. It can be reached by descending trail A to the water's edge, going upstream to a rift, and climbing up this rift for about 1,000 feet of vertical distance until the shoulder above Fosters Slide is reached. From there the descent to the slope must be made by climbing down the vertical face of

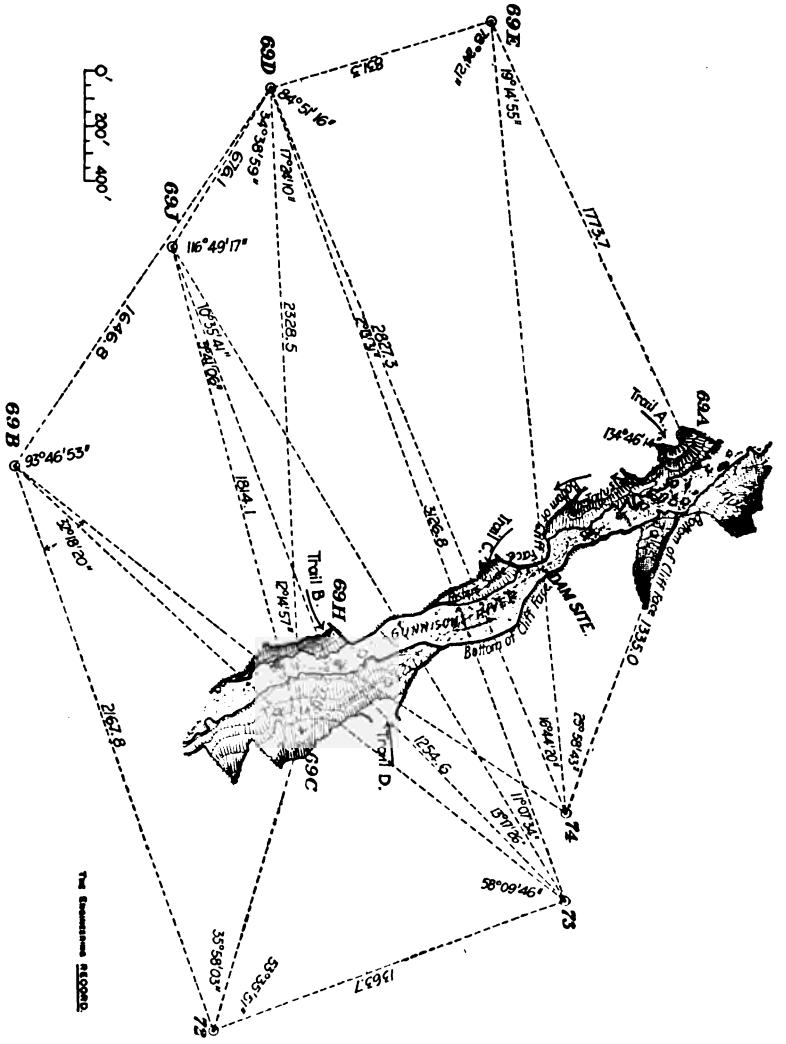

FIG. 1.—Diagram showing method of survey of Gunnison Canyon.

the cliff, the final 300 feet being almost perpendicular, to be overcome by the aid of a rope anchored at the top. Trail D leads on to the main talus slope on the north side of the river. Trails A and B are not dangerous, requiring only a due amount of hard work and caution to traverse them. Trail C, as noted before, is a difficult trail, and can

be passed over by none but the best climbers, men strong of limb and of steady nerves. It would be an impossibility to carry instruments into the canyon over this trail. Trail D is also a difficult trail. In many places it must be traversed with extreme caution. Over 300 feet of rope was left in this trail as an aid in scaling the vertical drops. Throughout almost the whole length an incautious step may send the climber rolling over precipices to certain destruction, or a careless move may loosen bowlders, to go crashing down upon the party below.

In very few places can the bottom of the canyon be seen from the rim on the south side, and triangulation stations were therefore established on the north side of the canyon. From these north side stations it was found possible to locate signals in the bottom which could be joined into the main triangulation scheme. Signals 69A and 69H are in the bottom of the canyon on the south side of the stream. Signal 69C is in the bottom of the canyon on the north side of the stream. Signals 72, 73, and 74 are on the north rim of the canyon. All other signals are on the south rim of the canyon. The triangulation plan will show the observed angles and the computed distances.

In sketching contours, stations 69C, 69H, and 69A were located, and from these a secondary system of triangles was laid out in the canyon. Contours in inaccessible places were then located by means of intersections, in accessible places by means of direct stadia readings, the work being plotted on the plane-table sheet as fast as taken. This detailed topography is on a scale of 50 feet to an inch, with a contour interval of 5 feet. The topography on Foster's Slide was taken by transit intersections from a base located on the main talus slope on the north side. In order to get the proper points on the slide a party was sent down over trail C to set signals. Levels were carried into the canyon on the Fellows Traverse by leveling down trail A and by vertical angles.

On account of the excessive vertical angles and the somewhat dangerous nature of the work it was feared that the results might not be so accurate as would be desired. The topography was taken on three different sheets, and when the sheets were assembled a most gratifying agreement was found. The traverse line, the triangulation, and the plane-table work checked each other closely. Triangulation angles were read and doubled, direct and reverse, the vernier reading to 20 seconds. Instruments were not shaded from the sun. The largest error of closure in any triangle was 13 seconds. The levels checked by the various lines within 0.15 foot. Taking into account the short sights and the extreme vertical angles involved these results are considered fairly good. From stations 69H and 69C the vertical angles to stations on the rim were sometimes as great as 70 degrees, making the use of diagonal eyepieces necessary.

COLORADO RIVER.

By J. B. LIPPINCOTT.

The drainage basin of Colorado River, 2,000 miles in length, contains 225,000 square miles above Yuma, and includes portions of five States and two Territories. In area it is one-fifth that of the Nile Basin, and second in size among the rivers of arid America, the Missouri being first and the Columbia third. The upper two-thirds of the basin consists of high plateaus and snow-capped mountains, while the lower one-third includes the Grand Canyon of the Colorado and desert regions.

In the lower course of the river there is a series of canyon-like valleys from 5 to 15 miles in width, which annually are inundated by the high water of the summer floods. The grade of the river from Needles to Yuma is 1.4 feet per mile. Below Yuma the river has built up a delta, and by its great deposits of silt has cut off a northern arm of the Gulf of California, which is now dry land below the sea level, and is known as the Salton Sink.

The river was discovered by Alarcon, a Spanish explorer, in 1540, who sailed up the stream from the gulf for fifteen days, probably reaching the present site of Ehrenberg, Ariz. The first Americans who appeared on the lower Colorado were trappers in 1825, and the first scientific account of its lower courses was given by Maj. J. W. Powell, in 1869.

WATER SUPPLY.

The annual variations of the river have a marked constancy. The spring rise, caused by the melting of the snow in the high northern mountains, appears in the lower Colorado about March 15, culminates in June, and reaches a minimum stage in midwinter, when the northern portion of the basin is largely frozen up. The river thus furnishes the greater flow when it is most needed. In January the low-water flow is about 3,000 cubic feet per second, but during the June rise the discharge will probably average about 40,000 cubic feet per second. The river is believed to have been about normal during 1902, the estimated flow being 7,960,000 acre-feet, whereas the discharge for 1903 was 11,329,132 acre-feet, these being the only two years of complete record. It is estimated that the duty of water on the lower Colorado is 1 second-foot to 91 acres in July and 1 second-foot to 147 acres in September, though it is claimed that larger quantities are being used. September is the controlling month of the year, as the supply is then low and the duty high. The total depth for the year is taken at 5.5 feet. In September, 1902, with the above duty, there would have been sufficient water to have irrigated 561,000 acres.

There are 400,000 acres of irrigable land along the lower Colorado

Valley in the United States, requiring a continuous supply of 2,720 second-feet of water in September. In addition it is estimated that there is as much more irrigable land in the Salton sink or Imperial country. About 300,000 acres of the valley lands are above Yuma and seepage water returning therefrom will be available for the Yuma and Imperial projects. If we assume 50 per cent of the water applied as returning in a continuous flow these 300,000 acres would yield 1,150 square feet and irrigate in September 169,000 acres in addition to the 561,000 acres given above, or 730,000 acres in all. The seepage basis in the short canals for the valleys being less and the return waters therefrom being great they should be first supplied. On this basis there would be available for the lands near Imperial water for 330,000 acres, less such seepage losses as may occur, which may be as great as one-third to one-half. This indicates an urgent necessity for storage, ultimately, to meet the late summer drouth. This water will not have to be held for a greater period than one hundred days subsequent to the June floods. The amount of water required to be stored in order that all the lands may be irrigated will be about 500,000 acre-feet for August and September uses.

Two storage reservoir sites were surveyed by the reclamation service during last season. At one point a dam 75 feet high will impound 400,000 acre-feet of water. At another a dam 75 feet high will store 1,300,000 acre-feet, making 1,700,000 acre-feet for the two, and, if they are built, all the irrigable lands in the United States along the Colorado River may be reclaimed. Explorations made at these points to date indicate that the bed-rock conditions are not satisfactory. These are not the only reservoir sites known to exist along the Colorado. Other sites are situated higher up the stream, but are not yet surveyed. As the Colorado River carries a high percentage of silt, the life of these reservoirs would not be long, but the last-mentioned site would hold all the silt that the river carries in forty years. These reservoirs are situated where the valleys are broken and where the land is not well adapted to agriculture.

NAVIGATION.

Prior to the construction of railroads to the lower Colorado, the river afforded the only feasible means of entrance into the desert regions of southeastern California and western Arizona, and therefore great efforts were made to navigate it. Subsequently, however, navigation on the river practically ceased. The shifting, shallow character of the bed of the stream prevents the maintenance of a permanent channel, particularly during periods of high water and when the river is falling. The improvement of the river for navigation has been considered on several occasions by army engineers, and has always been condemned.

CLIMATE.

The temperature on the lower Colorado is popularly supposed to be almost unendurable in summer time, but our best authority on the climatic conditions of the arid regions, Prof. Willis L. Moore, Chief of the Weather Bureau, in a most interesting communication which he presented at the fifth annual session of the National Irrigation Congress, held in Phoenix in 1896, demonstrates that as far as the effect of summer heat on the human system is concerned, Yuma is more favorably situated than Topeka, Cincinnati, Washington, Baltimore, Philadelphia, New York, or localities south thereof. He reaches this conclusion by showing that the true effect of temperature on the human system is measured by the wet bulb rather than the standard thermometer, and that the rapid rate of evaporation in this desert region reduces, by cooling, the effect of heat upon the bodies of animals. During eight months of the year this desert climate is very pleasant. The rainfall at Yuma is about 5 inches, the average annual temperature with dry bulb thermometer is 72 degrees, and the humidity is 43 per cent.

SILT.

Prof. R. H. Forbes, of the Tucson Experiment Station, Arizona, is the best authority on the character and value of the silt and minerals in the waters of Colorado River. The United States Geological Survey is now cooperating with him in these observations. He finds that the silt comes from the lower third of the drainage basin, and ranges from 62 parts per 100,000 in January to 374 parts to each 100,000 during the summer floods. The maximum amount of silt occurs in the water that enters the river after the summer thunderstorms of Arizona, the amount reaching 2 per cent dry weight.

Professor Forbes estimates that with a duty of water of 4 feet in depth annually there will be deposited one-tenth of an inch of solid soil each year, and that in one year the river discharges enough silt to cover 53 square miles 1 foot deep. The fertilizing values in the sediment are largely nitrogen, estimated to be worth as fertilizer $1 per acre-foot of water. The soils of the lower Colorado valleys are practically all composed of these river silts. There is also rolled down along the bottom of the river a continuous procession of sand. Consequently the river, in shifting its channel from one point to another, leaves numerous deposits of sand throughout the finer silts. These sands will assist in draining the bottom lands when they are irrigated in the future.

There is about one-third as much silt in the upper stratum of the water of the flowing river as in the lower, indicating that diversions into canals should be by a skimming process from the surface of the

river, aided by flushing the deposits that occur in basins at the head of the canal. The silt problem is a difficult one, and will have to be met, but it is believed that it should result in a benefit rather than an injury. The slit is exceedingly fine, plastering the sides of the canal as smoothly as could be done with cement, and offering as little resistance to flow.

SALT CONTENT.

The amount of mineral matter held in solution in the river is relatively high in the low stage and low in the high stage. During the winter the salt will reach 100 parts in 100,000, and in the summer flood stage 32 parts in 100,000. At no time does the river contain an excess of sodium carbonate (black alkali), but it does contain extensive quantities of calcium sulphate, which is beneficial to the land, and a neutralizer of the carbonates. Both the sediment in suspension and the salts in solution in Colorado River are pronounced beneficial for irrigating purposes, and it is believed that they will relieve alkaline conditions rather than aggravate them in lands bordering the river.

IRRIGABLE LANDS.

Along Colorado River and below the mesas it is estimated that there are 400,000 acres of irrigable lands. There are at present, in addition, approximately 500,000 acres commanded by the Imperial Canal System, making within the United States at least 900,000 acres of land that may be reclaimed by this great river. In Egypt there are 6,250,000 acres of irrigable lands in the valley and delta of the Nile.

CROP VALUES.

Under the Imperial Canal, which covers the Salton Sink from the lower Colorado, a first charge of $30 per acre is made for a water right, and an additional $2 per acre annually. A charge of at least 50 cents per acre is made for lands under the canal for which a water right has been purchased in case the water is not used. In the district below Yuma a start has been made in irrigating some of the river bottom land, which yields seven crops of alfalfa in a year, averaging 1½ tons per acre for each crop, worth annually $50 per acre. With grain it is customary to grow a crop of barley during the spring months and follow this on the same land with a crop of maize or corn the same year. Millet, kaffir corn, and alfalfa are particularly suitable to this locality. Stock raising is probably the best natural method of disposing of standard crops. Experiments are being made with cotton, date palms, and other semitropical crops. On the mesa lands the soil and climatic conditions are fitted for vegetable and fruit growing, including citrus varieties, which ripen approximately six weeks

earlier than like crops on the coast near Los Angeles. It is believed that with a substantial irrigating system these lands should become worth $100 an acre within a short period of time.

WORK OF RECLAMATION SERVICE.

The first irrigation investigation along this river for the Government was made by the writer in the winter of 1901 and 1902, and surveys were begun with vigor in the fall of 1902. Mr. E. T. Perkins is engineer in immediate charge. Triangulation and precise levels have been extended from the Mexican line to the canyon, a distance of 400 miles. Topographic maps upon the scale of 2 inches to the mile and with a 10-foot contour interval have been made for this entire distance. These surveys show relative elevations, reservoir sites, irrigable lands, and overflow conditions. Diamond-core-drill work has been done at four of the reservoir sites. The flow of the river has been measured continuously at Yuma, and records have been kept of the amounts diverted by canals. Evaporation has been observed, and silt values have been determined.

An effort has been made to study this river as a whole and to so plan the work thereon that each project may be in harmony with a complete scheme for its utilization. The public lands along this river were withdrawn upon the passage of the reclamation act, pending these surveys, to the extent of 3,000,000 acres. There have also been withdrawn for reservoir sites 350,000 acres.

CONDITIONS IN IDAHO.

By Governor JOHN T. MORRISON, of Idaho.

Idaho, the State which I represent, is deeply concerned in the matter of irrigation, and is especially interested in the work of the reclamation service. I realize fully the importance of the work of this bureau of the Government, and shall, as far as possible, cooperate with the engineers of the service engaged upon the work in my State. To that end I invite a greater comity. The State engineering and land departments, in the discharge of their respective duties under our laws, are daily confronted with problems which have close relationship to the work of the Government. Their action, in many instances, should be, and I am sure would be, guided in important particulars were a knowledge of the Government's plans available. In many instances to act without such comity will lead to future embarrassment and will curtail the useful possibilities of the bureau. You will appreciate that Idaho is a new State and that there are many problems coming up to the departments for original settlement. Very many of these are related to the problem of irrigation.

I realize that the future agricultural development of a very large part of Idaho depends upon the development of irrigation in the State and that with these matters your bureau is directly concerned. The action which your bureau may or may not take in connection with any particular enterprise, looking to the development of a section of the State from an irrigation standpoint, has a direct bearing and influence upon the action which the State through its departments may or may not take in connection with enterprises directly related to yours. We know that your work will be of inestimable value to the State and are therefore very much interested in its progress.

The reclaimable area in Idaho is, I believe, larger than that in any other State in the arid belt, and we have an abundant water supply, if properly conserved, to meet the requirements. I believe Idaho is the best watered State in the arid belt. The field is a most inviting one from an engineering point of view, and I am glad to congratulate those who are employed in the service in Idaho upon the splendid opportunities for marked achievement.

There is one point which I would emphasize. We need the help of the Government in those sections where the human element of the problem is conspicuous. That is, where the State departments meet with embarrassment and where the greatest good can be accomplished. We have beautiful valleys, where climate and soil conspire to make ideal homes for men. Naturally these valleys have attracted settlement, and to some extent, under present conditions, the population is already congested. Large investments have been made and splendid properties have been developed. These valleys are susceptible of wider cultivation, and the promise of profit is most inviting. There is one drawback—the available water supply is limited in comparison to the opportunities for its use. In consequence annoyance and friction has arisen in some communities, and our courts and departments of State are embarrassed by this. This is where we need help. Reenforce our water supply for those communities where homes are established and property interests are involved. I believe this to be more important than any other phase of the work under consideration by your bureau, and certainly the interests of the people whose property rights are involved claim attention rather than new enterprises in territory not yet settled. I do not doubt the splendid opportunities in the new fields nor discredit the investigations being made. They look to future development, and quite properly. I ask help for the pioneer who is struggling with the developed problem of the present.

In conclusion, let me say we have the keenest interest in your work, and look with ever-increasing hope upon all you do. I invite you to Idaho, and assure you of a sincere disposition on the part of our people to cooperate with you. I thank you for the courtesy extended in asking a word from me.

INVESTIGATIONS IN IDAHO.

By D. W. Ross.

Idaho has an area of about 84,000 square miles, of which about 65 per cent, or 35,000,000 acres, the southern portion, is arid. With the exception of a small area lying in the Salt Lake Basin, this arid portion is within the drainage basin of Snake River, so the future of agriculture in this part of Idaho—in fact, the future of the State itself—is linked with the possibility of irrigation development in the Snake River Valley.

The area drained directly by this stream is about 13,000 square miles. Several large areas are drained by streams that were evidently at one time tributaries of the Snake, but the courses of these are intercepted by the great lava flow, which fills and levels up the valley over an area of nearly 12,000 square miles. All of the run-off that is not used for irrigation in the upper valley of these streams sinks below the surface near the outer edge of the lava fields, disappearing completely, but reappears again in the form of great springs which break out from the sides of the canyons of the Snake River more than 100 miles below. Thousands of acres of this vast plain have a most fertile soil of great depth, and could be easily irrigated if water could be brought to them.

Snake River rises in western Wyoming, and flows in a westerly direction through this valley, having cut its channel into the great floor of lava to a depth in places of nearly 1,000 feet, its work being marked by canyons and waterfalls, all on the most magnificent scale imaginable. The work of developing the agricultural resources of this valley is in some places extremely simple, in others very difficult. Up to the present time development has moved along lines of least resistance, but practically all has been accomplished that can be done by communities acting through the methods usually employed, and everything that can be safely left to the efforts of private capital, actuated by the hopes of large profits, has also been done. The former agency has about reached its limits, on account of the magnitude of the undertakings themselves; the latter for the reason that sound public policy demands the carrying out of plans broad enough to provide for the needs of vast interests in addition to, but widely separated from, the projects directly involved, in order that serious conflicts between communities having interests in common may be obviated, and the endless complications which follow in the wake of irrigation extension by this agency may be prevented. So the chance of enlisting the assistance of the Government in the work of further development of this State comes at a most opportune time.

The largest bodies of irrigable public lands in Idaho are situated in the eastern, central, and western portions of Snake River Valley.

The reclamation of these lands requires solution of serious engineering problems involved in the construction of the necessary works, besides necessitating the almost complete control of the discharge of Snake River and nearly all of its principal tributaries. The area of these tracts and of those already partly reclaimed is so large that the storage facilities of this great drainage system will be taxed to its limits, while the consummation of all the plans will require the expenditure of many millions of dollars.

Investigations made with a view to the reclamation of public lands have been in progress under the provisions of the reclamation act for more than a year. More than 1,600 miles of preliminary surveys for canals and reservoirs have been made during this time, and work is now being energetically pushed forward. The surveys made to-day show the entire feasibility of reclaiming about 600,000 acres of desert land, 400,000 of which are public, and of controlling the flood discharge of nearly all the principal streams used for irrigation. Estimates of the cost of constructing the necessary works are being prepared as rapidly as possible and will all be submitted to the Department before the close of the year.

The economic benefits that may result from the reclamation of arid lands depend very largely upon the location and area of the tracts to be reclaimed; that is, whether a dense population will some day be sustained and centers of wealth and influence be established, or whether scattered and sparsely settled and small communities only will result. Idaho, from the standpoint of an arid country, is remarkably fortunate in the nature of her irrigation possibilities. Practically all the reclaimable land is situated at an altitude where farming is extremely profitable and where the value of land, based upon its products alone, is already very high. The climatic conditions, the extreme fertility of the soil, and the splendid water supply promise a dense population over all the large areas to be reclaimed.

The upper Snake River Valley contains more than 700,000 acres of irrigable land, more than 240,000 of which has been reclaimed since 1890. This land is in one solid body, all of excellent quality, and all easily irrigable. In order that this figure may not appear mean and small to those who, during the past few years, have been feasting their imaginations on reclamation projects involving nothing less than millions of acres each, I will state that the area mentioned is equal to one-half of all the land now irrigated in the State of California, to nearly one-half the amount irrigated in Colorado, and more than is now irrigated in any of the other States and Territories except Montana. The Boise, Payette, and Weiser valleys, contiguous districts in Idaho, contain over 450,000 acres of irrigable land—a larger area than is now irrigated in many of the arid States. The central portion of the Snake River Valley contains about 400,000 acres of

irrigable land lying practically in one body, and there are other large tracts well situated that are full of promise for the future so far as population and wealth are concerned.

In this brief outline of the conditions surrounding irrigation in Idaho and of the work which is being done for its extension under the provisions of the reclamation act, it is not possible to state definitely the amount of land that may be gained for agriculture by irrigation. On a conservative estimate it is very great. There are not, however, 15,000,000 acres of irrigable land in the Snake River Valley, as many of our citizens are in the habit of stating. It will perhaps never be practicable to irrigate more than 3,000,000 acres, as the amount of water available will not supply more than that area. But there are probably more large bodies of first-class irrigable land in Idaho than in any other State in the West, and when it is all reclaimed, and its great resources are properly developed, there is no reason why Idaho should not, in a very few years, stand in the first rank of the western States in population and wealth.

LIMITS OF AN IRRIGATION PROJECT.

By D. W. Ross.

Shall the flood or low-water discharge of a stream determine the capacity of a canal and limit the dimensions of an irrigation project?

Normal development of irrigation.—Like all undertakings prompted by selfish interests, irrigation development in most of the arid States has moved along lines of least resistance. The pioneer settlers, who were few in number, usually occupied lands along the first bottoms of streams, and diverted the water for irrigation, in many cases by constructing simple brush dams, sufficient to raise it nearly to the level of their lands, to which it was usually conducted by a ditch built at trifling cost. Other settlers, who followed, reclaimed small tracts by means of the same simple works, until all the land that could be reached in this inexpensive way was placed under cultivation.

But still other settlers continued to arrive, demanding homes, and willing to overcome greater obstacles than those met by the pioneers. Canals were constructed that led the waters out onto the lower bench lands, and these were gradually enlarged and extended. But there still remained, in many cases, the broad acres of the upper benches, to reach which required the expenditure of vast sums of money to pay for blasting channels through solid rock or constructing, in many places, miles of expensive flumes along steep canyon walls. The individual, and in most cases the community, lacked sufficient strength to overcome these or other natural difficulties and, as a result, irrigation development came to a standstill in the locality.

Questions to be considered by the engineers of the Government.—While the irrigation development already accomplished by individual effort, especially by the landowners, really furnished the foundation of civilization in the West, it is the execution of the greater projects—the ones beyond the means of the agencies mentioned—that is regarded as of greatest importance to the community, and these are the undertakings which in most of the arid States and Territories the Government is expected to take up and carry to completion. It is the problems connected with these enterprises that the engineers of the reclamation service are now required to consider. These problems are as varied in nature as the variable combinations of the physical features of the enterprises themselves, ranging from the adjudication of the rights of a water user to the utilization of a storage site. In fact, the legal and economic questions are likely to be much more important in many cases in determining the feasibility of a project than are the engineering problems. In no case would it be safe to decide as to the feasibility of any undertaking without first ascertaining what relation it will bear when completed to all other enterprises of a like nature in the locality.

Practically all the questions which have to be considered in connection with undertakings of this kind are raised when the subject of water supply is considered, and two of these questions—the ones most intimately connected with all others, both economic and engineering, namely, the capacity of the canal and the magnitude of the undertaking—form the subject of this discussion.

The conditions under which these questions will be considered are the average conditions existing in connection with many of the projects now under consideration by the Government, especially in the Northwestern States, and are as follows:

(1) A stream of great flood flow; (2) an area of desert land that may be put under canal out of all proportion to the water supply available during the latter part of the irrigation season; and (3) storage facilities, if they exist at all, entirely inadequate for the purpose of controlling the natural flow. Under such conditions the question is, What other factors must be considered, and what weight shall be given to each in fixing the capacity of the canal? Which means, in the end, fixing the limit of the undertaking.

Climate and range of products.—Knowing the nature of the available water supply, we should next consider the climate, the length of the growing season, the kinds of crops grown, the average date of harvest, the number of cuttings of hay that can be made during each season, and the part of the growing season in which occurs the flood discharge of the stream whose waters are to be used. It happens that the flood waters of the streams of most of the States and Territories

can be utilized for irrigation during some period of the growing season, generally at the beginning. In altitudes so high that but one cutting of hay can be grown, this flood discharge is usually available during the maturing of both hay and grain. When two or more cuttings can be made, it is generally available for the first and second, and for the irrigation of all kinds of grain crops, but may not be available for the third cutting of hay nor for the irrigation of a large acreage of orchard land.

In places where the climatic conditions admit of but one cutting of hay the problem is very simple. The irrigating season would begin early and be short, all or nearly all of the waters being used during a period of perhaps sixty days. In such cases, however, water must be used generously, and since practically all the land would require irrigation at the same time, the capacity of the canal must be very large. Assuming that the area of irrigable land is greater than the stream can serve, the average discharge of the stream during the period mentioned—provided it is regular in its habits—is the volume that should be diverted, and this factor will fix the limits of the undertaking, both as to the size of the canals and the area to be reclaimed. The exact capacity of the canal will depend upon the average depth of water required each day on the land to be reclaimed during the period of greatest use.

Value of flood water.—When two or more cuttings of hay can be made from the land, and when there is a great disproportion between high and low water, both stages occurring during the growing season, and when the storage facilities are entirely inadequate the question becomes complex. A water right in most cases is understood to be the right to use water during the entire growing season of crops, and land values will be found to be based upon this kind of a right. The question to be decided in fixing the capacity of the canal under such conditions is, in reality, What kind of a water right shall be given to the land? Shall it be given a "full season's right" or shall it be a "flood right"? which means: Shall we limit the area of land to be developed to the unused portion of the low-water discharge of the stream or shall it bear a certain relation to the flood discharge?

A great many people have an idea that by some means or other the Government will be able to increase the water supply; and that they may settle any lands under irrigation with the certainty of being supplied. It is contended, therefore, that it is not good policy to invite settlement unless the water is adequate throughout the entire irrigating season; in other words, it is contended that no more land should be put under canal than can be supplied during the entire season. This sounds very safe and conservative, but I do not believe it is a broad view to take of the possibilities of irrigation under the reclamation

act. The question to be considered is, not what is the most conservative plan for to-day, but what policy will lead to the development of each locality to the full extent of its possibilities. In undertakings proposed to be carried out by the Government the line of least resistance will not be found in the "conservative" plan referred to, but will be found in the scheme that is so broad and comprehensive that future generations will not think of supplementing it.

One important factor in determining the limit to be placed on the magnitude of undertakings of this kind is the value of the land after it has been put under irrigation. Will the land, after its irrigation, be worth the expense of its reclamation? The consideration of this question leads us to inquire as to the kinds and quantity of crops that can be produced on the land during the different periods of the growing season, and finally resolves itself into a question relating to the value of the crops produced rather than to the length of the irrigating season.

If the value of the crops that can be produced by early irrigation, for example, the first and second cuttings of hay, or a crop of grain, gives a value to the land greater than the cost of its reclamation, the project should be regarded as feasible, though other lands in the same locality may happen to have a water right which makes possible three or four cuttings of hay or renders the adjacent highways impassable by mud during the month of August.

If we must decide as to the capacity of a canal under the conditions stated, and have nothing more on which to base our judgment than the fact that with certain climatic conditions we would be enabled to mature by a certain date a full crop of grain and one or two crops of hay, the magnitude of the project would be measured by the supply of water that would be available up to the last irrigation of such crops, and no attention whatever need be paid to the available supply of late water. For example, if the climatic conditions render it possible to make four cuttings of hay during the season on lands irrigated from a certain stream (the first and second cuttings as well as the harvesting of grain occurring before August 15) then, if the mean flow of the stream up to July 20, say, is 2,000 second-feet and the available supply during the months of August and September is 500 second-feet, the computation of possibilities of land reclamation from this stream should be based upon the former and not the latter amount, and the capacity of the canal in this case should be 2,000 second-feet, the number of acres to be irrigated being determined by the average quantity of water required by each acre during the period when most water is needed. If it is found that it will be necessary to apply water to an average depth of 1 foot during a period of thirty days over all the land to be irrigated the canal should have a capacity

of 1 second-foot for each 60 acres. The water supply would then be sufficient for the irrigation of 120,000 acres and the project should, under these conditions, be limited to that acreage.

Value of low-water supply based upon the utilization of the flood waters.—Making use of the foregoing example we will inquire into the efficiency of the late water, or the supply of 500 second-feet available for use during August and September. Of this tract of 120,000 acres fully 30 per cent, under the conditions referred to, would be cultivated to grain; about 50 per cent to meadow, and the remainder, 20 per cent, would be planted to orchards and gardens. The grain land, or 30 per cent of the total area, would not require irrigation after, say, July 20, which would leave 70 per cent, or 84,000 acres, to be provided for during August and September. With a canal of generous capacity, such as has been provided, the entire acreage could be thoroughly irrigated, and would be in the best possible condition at the beginning of the dry season. In fact, the most favorable conditions imaginable would exist for giving the small supply of 500 second-feet a high duty. This small supply would be prorated among the irrigators, each landowner having 25 per cent of his flood head, which must now do for the irrigation of 70 per cent of his land, or a duty of about 170 acres must be given for each second-foot. This extreme duty would probably not be attained in all cases, but if the land and crops were in good condition at the beginning of the dry season and every landowner had the same interests at stake there is no doubt that a very high duty could be reached in a few years' time, and that finally every acre of the 70 per cent would be producing a full crop up to the end of the growing season.

Results following the use of low water alone.—Now, what would be the results following a more "conservative" plan, one in which the area to be developed is based on the supply of 500 second-feet available during the low stage of the stream? Under this plan the area to be developed would probably not exceed 50,000 acres, which would require a canal having a capacity of 1 second-foot for, say, each 100 acres of land. With this small capacity per acre, if the principal crops were hay and grain, the greatest economy would have to be practiced from the very beginning of the irrigation season, and it is not likely that during the period of greatest use enough water could be delivered to insure the best results. It is extremely doubtful whether the average yield per acre would be as great as if a canal with greater capacity had been provided. Indeed, it is doubtful whether the results of a full season's irrigation under these conditions would be as profitable, acre for acre, as those obtained by early irrigation alone under the plan first considered. One thing is quite certain, neither the soil nor crops would be in as good condition at the beginning of the dry season. The temptation under this plan is to arbitrarily fix the duty of water

by limiting the size of the canal. The effect of this plan is usually just the opposite of what is expected. I have never known nor heard of a canal with a capacity too great, but hundreds of them have been built too small for the lands dependent upon them. Nothing is more exasperating to the irrigator than to be restricted in the use of water at a time when oceans of it is wasting down a stream from which his canal is diverted. If the mistake is his own, he remedies the defect by enlarging his ditch. If the canal belongs to a company, he has been known to construct one of his own according to his own ideas.

Conclusion—In conclusion, a word might be said regarding the breadth or comprehensiveness of these plans. The difference between the two plans outlined is this: The first provides for the irrigation of the maximum acreage upon which it is possible to produce one full crop each and every season, which is more than can be counted on for a certainty on 90 per cent of the agricultural land of the United States. We provide a canal with large capacity for the irrigation of this land, because the irrigation season will be short and every acre must be supplied during this period. This provision insures the largest possible returns from grain crops and the first cutting of hay, and produces a favorable condition of soil at the approach of the dry season, thus minimizing the possibility of loss during that period. The same quantity of water will be available during the latter part of the season under this plan as under the other, but all the land will be in first-class condition at the beginning of the period of low water and the highest possible duty of water can be attained; which means that while more than double the area will be irrigated during the early part of the season, a much greater area will be served during the period of low water. The first plan is the plan for all time. All will have been accomplished that will ever be possible under the natural conditions which exist and which can not be changed.

Under the second plan we dedicate the use of the low-water supply to a limited area, just as the early settlers have done in every valley throughout the West. This will become a vested right, and the use of this limited supply of water will not, as a result, be extended to other lands. It is true these lands will have a "first-class water right," but a much larger area under the first plan would have as good a right with as much more in addition thereto rescued from the desert, capable of producing one full crop every season, an area which under the last plan would likely be condemned to everlasting sterility.

One good crop a year is far more than the average returns from the labor of the agriculturist, taking the world over. This, in my opinion, would be a very conservative basis upon which to estimate the possibilities of a reclamation project and should fix the limits of our plans and expectations. With these limits fixed, the flood waters should be utilized first, and upon the wealth thus created we should levy

for the construction of storage works, if the proper facilities exist. If such facilities do not exist, then we shall have fully utilized the resources at hand and left no complications or problems to be solved by the next generation.

INVESTIGATIONS IN MONTANA.

By Cyrus C. Babb.

MILK RIVER PROJECTS.

NATURAL CONDITIONS.

In the northern part of Montana, close to the international boundary, is a section of mountainous country unsurpassed for its scenic beauties. Here are peaks covered with snow the year round, and a little lower down there are glaciers, the largest in the United States. Numerous streams, receiving a plenteous supply from the heavy winter snowfall in this basin, descend the slopes of these mountains and their waters are collected in St. Mary Lakes, which are drained northward by the river of the same name into the Saskatchewan, and are finally lost in Hudson Bay.

Milk River, heading in the low, rolling country lying immediately east of this basin, runs in a general northeasterly direction, the principal branches, North Fork and South Fork, uniting after crossing the Canadian line. The stream thus formed flows eastward for 150 miles or more, when it bends southward and again enters Montana, finally emptying into Missouri River. The broad Milk River Valley in Montana consists of gently rolling country, well adapted to irrigation. The water supply from Milk River, however, is deficient, owing to lack of high mountain catchment area at its head.

The Milk River or St. Mary project is designed to store flood water in the St. Mary Lakes and conduct it eastward by a canal cutting through gravelly ridges to the head of Milk River, so as to connect a mountain catchment area with that stream.

In 1900 a reconnaissance of the St. Mary country was made, and has been followed, during the last three years, by more detailed surveys. The investigation has brought out four projects for consideration:

1. Carry water from the St. Mary Lakes to the North or South Fork of Milk River, and allow it to run through Canada to the lower Milk River Valley in Montana.

2. Carry the St. Mary Lakes water in a long canal across country, keeping it in the United States for the entire distance, to Sage Creek, a tributary of lower Milk River.

3. Make the project a more local one; that is, irrigate the lands of the eastern portion of the Blackfoot Indian Reservation and those immediately adjacent eastward.

4. Carry water from St. Mary Lakes across both the North and South Fork of Milk River to Cutbank Creek, down which it will flow for 50 miles or more to Marias River, and then divert the latter and carry water in an artificial channel again to Big Sandy Creek, a tributary of lower Milk River.

DESCRIPTION OF WORKS.

Under the first project, it is proposed to build a low earthen storage dam at a point about three-fourths of a mile below the present outlet of lower St. Mary Lake. This dam will have a maximum elevation of 55 feet above the bottom of the river, and will form a reservoir having a capacity of 250,000 acre-feet.

During the year 1902 an examination was made at the dam site to determine the depth at bed rock and the character of the rock. Sandstone rock outcrops at the surface at the eastern end of the dam, whence it drops rapidly to 138 feet below the surface at 1,600 feet from the eastern end. Borings this year show a depth of 187 feet at one point.

The top of the earthen dam will be at an elevation of 4,513 feet above sea level; the total length of the earth embankment will be 2,650 feet; both inner and outer slopes will be 3 to 1. It is the intention to use the hydraulic-fill method of dam construction. Most of the material will come from the hill west of the dam, as the water to wash it down can be easily obtained from Swiftcurrent Creek by means of a short diversion canal. A concrete-core wall will extend from the east abutment of the dam across the river for a distance of 475 feet, where the present depth of bed rock below the surface of the ground is 33 feet. From this point to the western end of the dam a puddle trench will be first excavated and sheet piling driven as far as practicable. This trench will then be filled with selected material by the hydraulic method, and the earth to form the remainder of the dam will be deposited on both sides. The lower toe of the dam will be of riprap in order to prevent the sliding of the material due to any percolation of water under the dam. The upper face will be riprapped as a protection against wave action.

From the reservoir thus formed water will be diverted through an open cut in rock around the eastern end of the dam. The head gates of the reservoir outlet will be located opposite the end of the dam and connected with the concrete core wall. These head gates consist of a curved wall of 30 feet radius placed on four concrete piers. In this wall there will be two series of circular openings; an upper series, consisting of three openings 20 feet below the top, and a lower series, consisting of four openings 30 feet below the top, each opening being 4 feet in diameter. There will be five additional openings, each 5 feet

wide by 9 feet high, at the bottom or between the piers. Water passing through all the openings will be controlled by cast-iron sluice gates operated by proper gearing.

In the canal, 850 feet beyond these reservoir gates, there is to be a concrete spillway for discharging the waste water back into the river by suitable regulating gates.

DESCRIPTION OF CANAL.

The canal proper will begin immediately below the head-gates. The grade was assumed at two one-hundredths of 1 per cent, or a fall of 1.056 feet to the mile. It is proposed to make the bottom width 30 feet, with a 10-foot depth of water, and with side slopes of 1 to 1. It is assumed that in a canal of these dimensions the mean discharge at the head will be 1,350 second-feet. From the dam the canal will continue down the east bank of the river for a distance of 7 miles, then turn eastward and pass through a gap known as Spider Lake Gap. Thence it will continue in a general northeasterly direction for a distance of 27.4 miles, or to the North Fork of Milk River. The last step in the construction of the canal to reach the North Fork of Milk River will be the dropping of the water 180 feet to the level of the creek.

The following table gives the estimated cost of construction of this portion of the line, the water to be dropped into the North Fork as described:

Cost of St. Mary dam and canal from head to North Fork.

Earthen dam, outlet canal, and head-gates	$250,000
Head to Spider Lake	245,100
Spider Lake to drop at North Fork of Milk River	288,400
Drop at North Fork	16,040
Two sets of waste gates on line	4,000
Engineering and contingencies	120,530
Total	924,070

LOWER MILK RIVER.

Milk River crosses the international boundary, flows for about 150 miles in Canada, and comes back into the United States at an elevation of 2,680 feet. It continues in a southeasterly direction for about 60 miles, whence it bends eastward, which general course it holds to about Hinsdale, where it again bends southward until it reaches the Missouri. At Havre the elevation of the river is 2,467 feet, showing a fall from the international line of 213 feet in the distance of 60 miles, or a fall of 3.7 feet to the mile. This portion of the stream is in a deep valley, ranging from 1 to 2 miles in width, with bluffs on either

side about 200 feet high. A canal taken out at the international boundary at the grade of the river would continue down for about 60 miles until Havre was reached, and then would not be out of the bluffs. Throughout nearly all of this distance it would be carried along sidehill, and in view of this fact and the great length of the canal the proposition is hardly feasible. Ten miles below Havre the valley widens and here the first diversion of Milk River can be accomplished.

At the present time (1903) a survey is being made for a canal heading at Yantic, on the north side of the river, and continuing downstream for 60 miles. This will cover about 50,000 acres, a portion of which is at present irrigated from private ditches taken from Milk River. This canal will be a later development of the one described below, and no estimates of cost have yet been made.

MALTA PROJECT.

The original plans for the St. Mary Canal, made at the end of the field season of 1901, contemplated a system of storage in the lower Milk River Valley, and investigations were made here in 1902 with this object in view.

Eight miles east of Malta there is an alkaline body of water, known as Lake Bowdoin, having an area of 6.8 square miles. The lake lies in a depression that is surrounded by hills. A survey was made here with the idea of using the lake site as a possible reservoir. The plan contemplates a diversion of Milk River from its south side at a point 23 miles above Malta, whence the water will be carried to the reservoir.

The feed canal will be 34 miles long and is estimated to carry 2,000 second-feet of water. It will have a bottom width of 50 feet, a depth of 12 feet, a slope of 1.5 to 1, and a grade of 0.0001, or about 6 inches to the mile.

Between Malta and the reservoir there is a divide 50 feet above the lake. At this point a drop of 25 feet is made to the top surface of the reservoir, as only 25 feet of water is to be stored, but from this drop a high-line lateral will be carried southward around Beaver Creek, and irrigation lands in the valley of this stream.

For a storage depth of 25 feet, the capacity of the reservoir will be 298,900 acre-feet, and the surface area will be 15,000 acres.

In order to create Lake Bowdoin reservoir, it will be necessary to build a series of earth embankments across various gaps—eight in all. The total length will be 19,050 feet and the total quantity of excavation will be 599,961 cubic yards. The top width of the canal will be 10 feet, the slope on water face will be 2.5 to 1, and on lower face 2 to 1. Water will be drawn from this reservoir and served to the lands lying eastward, both north and south of the river.

Engineering surveys are being prosecuted and will be practically completed on this project this season. Topographical surveys are also being made on two different scales, one the regular field scale of the survey, that is 1: 45,000 or 1 inch to 3,750 feet, and a larger one, 1 inch to 2,000 feet, that will show all the agricultural lands that can be covered by the project, which is now estimated at 250,000 acres.

CONSTRUCTION IN NEVADA.

By L. H. Taylor.

In Nevada the work of the reclamation law has advanced to a point where construction has already been begun. The preliminary work has been described in the First Annual Report of the Reclamation Service, so that it is unnecessary to go into details in this matter. It is, however, of interest to all to know the conditions under which the work is being done, and for this reason special attention is called to the specifications and bids for construction of the Truckee canal, taking water from Truckee River to the lower Carson reservoir site.

There is presented here a copy of the original advertisement, the specifications, abstract of the bids, and copy of the agreement made with one of the firms of contractors. The work was advertised in three divisions, the successful bidders for the first and second divisions being C. A. Warren & Co., and for the third division the E. B. & A. L. Stone Company.

ADVERTISEMENT.

Department of the Interior,
Washington, D. C., May 15, 1903.

Sealed proposals will be received at this Department until 2 o'clock p. m. July 15, 1903, for the construction of about 15 miles of canals, with diverting dam, headworks, spillways, and tunnels, for the diversion and conveyance of 1,400 cubic feet of water per second from the Truckee River, between Clarks Station and Wadsworth, Nev., to irrigable lands in the Truckee and Carson valleys. Plans and specifications will be open for inspection, and blank forms upon which all bids must be made can be procured at the office of the Geological Survey, in Washington, D. C., and Reno, Nev., on and after June 20, 1903. Each bid must be accompanied by a certified check for 2 per cent of the amount of the bid, as a guaranty that the bidder will enter into a written contract with the Secretary of the Interior within ten days after the award is made, and furnish bond in the sum of 50 per cent of the contract price for the faithful performance of the work. The Secretary of the Interior reserves the right to reject any and all bids. Detailed information can be obtained on the ground from L. H. Taylor, resident engineer, Reno, Nev. Proposals should be addressed to the Secretary of the Interior, Washington, D. C., and endorsed on envelope "Proposals for Irrigation Work, Truckee River, Nevada."

E. A. Hitchcock, *Secretary.*

FORM OF PROPOSAL.

RECLAMATION SERVICE U. S. GEOLOGICAL SURVEY, TRUCKEE PROJECT—NEVADA.

Proposal for excavating main canal, and building headworks, spillways, sluice and wasteways, reduction chambers, tunnels, and highway bridge.

NOTE.—Bids will be received on one or more divisions or on all the work. Bidders desiring to bid on one division only will fill out their figures on printed schedule opposite that division only. Those desiring to bid on all the work will fill out the figures opposite all the divisions.

TO SECRETARY OF THE INTERIOR,
Washington, D. C.

SIR: The undersigned propose to do all the work, and furnish all the material, except the cement, in accordance with the printed form of general specifications, a copy of which is herewith annexed, and bind ————————, on the acceptance of this proposal, to enter into and execute a contract, with necessary bond, of which the inclosed general specifications shall be made a part, for the execution of said work at the prices named in the schedule below.

——— furthermore agree, that in case of ——— default in executing such contract, with necessary bond, the check accompanying this bid, and the money payable thereon, shall be, and remain the property of the United States.

GENERAL SPECIFICATIONS.

1.—Purpose of work.

The completed work has for its purpose the conveyance of fourteen hundred (1400) second-feet of water from Truckee River to the head of Pyramid Branch, a distance of about six (6) miles; thence the conveyance of twelve hundred (1200) second-feet of water to the lower Carson River reservoir, a farther distance of about twenty-four (24) miles.

2.—Plan of work.

The general plan of the work requires the construction of a concrete weir and headworks and earth embankment, also temporary dam and diversion channel on Truckee River about four (4) miles below Clarks station on the Central Pacific Railroad; two or more spillways or overflow weirs, one or more sluice and waste ways, headworks for Pyramid Branch, six or more reduction chambers, and one small bridge to carry highway across the canal, and excavating and completing the main canal, including the construction and completion of three or more tunnels, in accordance with the accompanying plans, which show general type of each structure.

3.—Plans and drawings.

The plans which accompany and which are made a part of these specifications are numbered and named as follows:

Drawing No. 1. Cross-section of main canal and tunnel.
Drawing No. 2. General plan for combined sluice and waste way.
Drawing No. 3. Details of gates for head works and waste ways, wrought iron.
Drawing No. 3a. Details of gates for head works and waste ways, cast iron.
Drawing No. 4. Plan for head works for Pyramid Branch.
Drawing No. 5. General plan for spillways in main canal.
Drawing No. 6. General plan for reducing chambers, canal to tunnel.
Drawing No. 7. Plan for head works for main canal.

Drawing No. 8. Plan showing location of head works and diversion of river during construction.

Drawing No. 9. Design for highway bridge crossing canal.

Drawing No. 10. Profile of final location of canal.

Drawing No. 11. Plan showing general location of canal; small scale.

4.—Special and general specifications.

Special specifications will be furnished the contractor for each structure as the work proceeds.

The general specifications shall apply to all work done or materials furnished, and shall control the special specifications where they are silent. In case of conflict between the general specifications and the special specifications the latter shall control in the particular work to which they apply.

5.—Manner of doing work.

All work shall be done in a thorough and workman-like manner, notwithstanding any omission from the drawings or specifications, and anything mentioned in the specifications and not shown in the drawings, or shown in the drawings and not mentioned in the specifications, must be done as though shown or mentioned in both.

6.—Engineer and contractor.

Where the word "engineer" is used in the general or special specifications or in the contract it shall be and is mutually understood to refer to the chief engineer of the reclamation service, United States Geological Survey, or any of his authorized assistants or inspectors, limited by the particular duties entrusted to them.

Wherever the word "contractor" is used in the general or special specifications or in the contract it shall be and is mutually understood to refer to the party or parties contracting to perform the work to be done under this contract or to their legal representatives.

7.—Time of commencement, and prosecution and completion of work.

On the acceptance of the proposal the chief engineer will give written notice thereof to the party or parties proposing personally or by letter or wire to their stated address; and in ten days from the date of such notice the contractor shall execute his contract, and bond by a surety company, and the work shall be begun promptly, with adequate force, according to the terms of the contract. The work shall be prosecuted continuously and vigorously, with such force and in such manner as to secure its completion within fourteen months of date of signing the contract. If at any time the work is not progressing satisfactorily the attention of the contractor shall be called in writing by the engineer to the fact that the progress being made is not such as to enable the contractor to finish the work in the specified time. On receipt of such notice the contractor must put on more force and make such showing as will satisfy the engineer that the work can be finished in the specified time. The amount of work done at the end of any month, except the first, must bear at least the same proportion to the total work to be done that one month does to the number of months fixed for the completion of the work.

8.—Putting on more force.

If the contractor, in a reasonable time after receiving such notice from the engineer, does not put on additional force, the chief engineer shall have the right to declare the contract null and void or to hire such force and buy such tools, at the contractor's expense, as may be necessary for the proper conduct of the work and for finishing it in the time agreed upon.

9.—Character of workmen.

The contractor shall discharge from his service, when required by the engineer, any disorderly, dangerous, insubordinate, or incompetent person. None but skilled foremen shall be employed on the tunnels and concrete work. As required by law, no Mongolian labor shall be employed on construction work.

10.—Methods and appliances.

The methods and appliances adopted by the contractor must be such as will secure a satisfactory quality of work and enable him to complete the work in the time agreed upon. If at any time such methods and appliances appear inadequate the engineer may order the contractor to improve their character or increase their efficiency, and the contractor must conform to such order; but the failure of the engineer to order such improvement of methods or increase of efficiency will not relieve the contractor from his obligations to perform good work or finish it in the time agreed upon.

11.—Measurements, estimates, and payments.

At the end of each calendar month the engineer shall make an approximate measurement of all the work done up to that date and an estimate of the value of the same at the prices agreed upon in the contract. A deduction of twenty-five (25) per cent shall be made from this estimated amount, and from the balance shall be deducted the amount of all previous payments. The remainder shall be paid to the contractor upon the presentation of proper vouchers endorsed and approved by the chief engineer. The twenty-five per cent deducted shall be retained by the Government until the work shall have been completed to the entire satisfaction of the chief engineer.

12.—Engineers' authority, and appeals, etc.

The engineer in charge shall decide as to the meaning and intent of any portion of the plans and specifications. He shall have the right to correct any errors or omissions therein where such errors or omissions might cause a misunderstanding of the meaning and intent of said plans or specifications. The Secretary of the Interior reserves the right, at any time during the progress of the work, to change the alignment, grades, and width of the canal, or any portion thereof, or to alter the character, vary the dimensions, or change the location of structures, or substitute one kind of material for another, or to omit entirely, where found necessary, or require to be built where not now contemplated; and the contractor shall carry into effect all such alterations, at the unit prices mentioned in the contract. Should such change, however, be made after the commencement of any particular piece of work, and result thereby in any extra cost to the contractor, the engineer in charge shall certify to the amount to be allowed therefor which he shall consider as fair and equitable between the parties, and his decision, when approved by the chief engineer, shall be final and conclusive.

13.—Inspection of work.

The engineers and inspectors appointed by the Secretary of the Interior shall at all times have the right to inspect the work and materials. The contractor shall furnish such persons reasonable facilities for obtaining such information as they desire respecting the progress and manner of the work and the character of the material.

14.—Quantities, approximate in schedule.

The quantities of the work presented in the schedule are only approximate, and the information given on maps and profiles as to summary of material is according to the best present knowledge.

15.—Removal of defective work.

The contractor shall remove at his own expense all material which is condemned by the engineer or his inspectors, and shall remove and rebuild at his own expense any part of the work which has been improperly executed, even though such work should have been already allowed for in the monthly estimates. The engineer shall give notice in writing to the contractor of such defective work, when found. If the contractor refuses or neglects to replace such defective work it may be replaced by the Secretary of the Interior at the contractor's expense.

16.—Protection of finished work.

The contractor will be held responsible for the material furnished to him, and for the care of the finished work until final completion of the same, and will be required to make good, at his own cost, any damage or injury it may sustain from any cause. He shall take all risks from floods and from casualities of every description, and make no charge for detention from such causes. He may, however, be allowed a reasonable extension of time on account of such detention.

17.—Damage and trespass.

The contractor will be held responsible for all damages to persons or property which may result from his acts or neglect, or that of his agents, employees, or workmen. He will not allow any of his agents, employees, or workmen to trespass upon the premises or lands of persons in the vicinity of the works, and will discharge, at the request of the engineer, anyone in his employment who may be guilty of committing damage in this respect.

18.—Delays.

The contractor shall not be entitled to any compensation for delays or hindrances to the work from any cause whatever. If such delay or hindrance is caused by failure to provide material sufficient to carry on the work, or failure to give such instructions as may be necessary for the same, on the part of the Secretary of the Interior or his engineers, then such delay will entitle the contractor to an extension of time equivalent to the time lost by such delay. The engineer must receive from the contractor notice, in writing, of such delay before allowing any extension of time.

19.—Responsibility.

The contractor will not assign or sublet the work, or any part thereof, without the written consent of the chief engineer, indorsed upon his contract. No subcontract shall, under any circumstances, relieve the contractor of his liabilities and obligations under the contract, and should the subcontractor fail to perform the work to the satisfaction of the engineer the contractor will be held responsible, and if he fails or refuses to make satisfactory the work done by the subcontractor the Secretary of the Interior may, at his option, end and terminate his contract.

20.—Extra work.

No claim for extra work or work unprovided for in the contract shall be considered or allowed unless previously ordered in writing by the engineer. If such extra work is ordered, payment will be made only upon the certificate of the engineer that such work has been satisfactorily performed, and demand for this payment must be accompanied by such certificate and made at the time of the next monthly payment after the extra work has been finished.

21.—Foreman and copy of plans, etc.

The contractor shall keep at all times upon the work a copy of the plans and specifications, so that reference may be made thereto by the engineer, in case of misunderstanding or misconstruction. Instructions given to the contractors, foremen, or agents on the work by the engineer shall be considered as having been given to the contractor himself.

22.—Roads and fences.

All roads crossing the canal must be kept open until proper bridges or crossings are provided, and all fences crossing the canal must be kept up by the contractor until the work is finished.

23.—Bench marks, stakes, etc.

All bench marks and side slope stakes must be carefully preserved by the contractor, and in case of their willful or careless destruction or removal by him, or any of his employees, such stakes shall be replaced by the engineer at the contractor's expense.

24.—Classification of material.

The classification of the material excavated shall be made by the engineer in charge, and his judgment will be final and conclusive in such matter, as well as in the inspection of material used in the structures.

25.—Right of way.

The right of way for the land necessary for the canal and for all borrow pits, channels, spoil banks, ditches, roads, etc., will be provided by the Secretary of the Interior.

26.—Maps and plans.

The engineer will stake out all the work for the contractor and furnish him, from time to time, with such detail drawings, plans, profiles, and special specifications as may be necessary to enable him to complete the work in a satisfactory manner. All such plans, profiles, maps, and drawings must be returned to the engineer on completion of the work.

27.—Unpaid claims.

The contractor shall pay all workmen employed on the work punctually, at least once a month, and also pay regularly for all material furnished under his contract. He will furnish the Secretary of the Interior with satisfactory evidence that all persons who have done work or furnished material and who shall have filed claims for the same with the Secretary of the Interior have been fully paid or are not entitled to any lien under the laws of the State of Nevada. In case such evidence is not furnished such amount as the Secretary of the Interior may consider necessary to meet the lawful claims of such persons shall be deducted from the money due the contractor and held until the liabilities shall have been fully discharged and evidence of the same furnished the Secretary of the Interior, and if such evidence is not furnished before the final payment under this contract falls due, the Secretary of the Interior may pay such claims, and charge the amount thus paid to the contractor, who shall accept the same as payment, to the amount of the claims on his contract.

28.—Failure to complete work in time agreed upon.

In case the contractor should fail to complete the work in the time agreed upon in the contract, or in such extra time as may have been allowed for reasonable delays

incident to the work, the engineer shall compute and appraise the direct damages for the loss sustained by the Secretary of the Interior on account of further employment of engineers, inspectors, and other employees, including all disbursements on the engineering account properly chargeable to the work. The amount so appraised and computed shall be deducted from any money due the contractor under his contract. The decision of the chief engineer as to the appraisal of such damages shall be final and binding on both parties.

29.—Slopes and batters.

Whenever, in these specifications and plans, the word "slope" or "batter" is used they will be understood to mean the ratio of horizontal to vertical. Thus a slope of three to one would mean three horizontal to one vertical, or a batter of 2 inches to 1 foot, 2 inches horizontal to 1 foot vertical.

30.—Bidders to examine ground before bidding.

Bidders must satisfy themselves of the nature of the material and of all local conditions affecting the work, and no information derived from the maps, plans, specifications, profiles, or drawings, or from the engineer or his assistants, will in any way relieve the contractor from all risks, or from fulfilling all the terms of his contract.

31.—Use and sale of liquor.

The use or sale of intoxicating liquor will be absolutely prohibited on the work except under the direct supervision of the contractor or his agent; and, if possible, such arrangements will be made with the local authorities as will prevent the issue of license to sell intoxicating liquors to anyone else than the contractor, at any point within 5 miles of the work where liquor is not now sold.

32.—General conditions.

The above general conditions shall apply to every part of the work to be done under these specifications.

33.—Grading.

Under this head will be included clearing and grubbing where required; all excavations and embankments required for the completion of the main canal or any of its branches, for all temporary work, and for permanent dam at the head works; all excavations for ditches or drains necessary for cross drainage; excavations for all foundations for bridges, culverts, or walls, and all excavations and embankments for road crossings, where they have been interfered with in the construction of the canal. All material moved in grading shall be measured in excavation only, and estimated by the cubic yard under the following classes. (Excepted herefrom is the excavation in water, at the head works, which is specially provided for in schedule.)

Class 1. All material that can be ploughed by an average eight-mule team, each animal weighing not less than twelve hundred (1,200) pounds, attached to a suitable ten-inch breaking plough; all well handled, by at least four men. Also all loose material that can, without ploughing, be loaded into a scraper by two men.

Class 2. Indurated material of all kinds, which cannot be ploughed as in class 1, but which requires loosening by powder, and can then be removed by scraper.

Class 3. All detached masses of rock, more than two, and less than ten cubic feet in volume, and all slate or other rocks, soft or loose enough to be removed without blasting.

Class 4. All rock, not included in the above classes, which requires drilling and blasting.

The canal will be excavated accurately to sections and forms similar to those shown in drawing No. 1, and to the lines, grades, and slope stakes, as established every 50 feet

by the engineer. The side slopes may be varied to suit the material, but in all cases will be staked out by the engineer. All material taken from the excavation will be deposited, when the canal is on sloping ground, on the lower side of the canal, unless otherwise directed by the engineer. When the canal is on level ground, the material from the excavation will be deposited on both sides, to form the upper portion of the waterway. If there is an excess of material in excavation on sloping ground, over and above that which is necessary to complete the waterway to the level given by the engineer, it will be deposited on the lower side of the finished bank, and on the same level with the top of the bank of waterway, and if there is an excess of material in excavation on level, or nearly level ground, it will be used to widen the banks on each side, as may be directed. All suitable material taken from the excavations must be placed where directed by the engineer, within a limit of average haul of 600 linear feet. All haul over 600 linear feet will be paid for at the rate agreed upon in the contract. Wherever banks are made on the side of the canal, to form part of the waterway, the ground under such banks must be entirely stripped of all brush and vegetable matter of every kind, and, if necessary, the roots grubbed out and burned with the other material removed. All work of this nature shall be included in the price paid for excavation. All banks, where required for forming part of the waterway, shall be built in layers of about 1 foot in depth, and to the elevations given by the engineer to allow for settlement. Where there is not sufficient material for banks inside the lines of the canal, the engineer will designate the place where such material is to be procured and will measure and allow for such material in excavation only, with allowance for overhaul, if any. Where any material suitable for use in structures, such as sand, gravel, or stone, is found in the excavations, it will be laid aside in some convenient place, if so ordered by the engineer. No extra allowance will be made for this work, if the material is deposited within 600 feet of the place where it is excavated. All embankments at or near structures shall be made at such times, in such manner, and of such materials as the engineer may direct. No banks intended to hold water shall be made in freezing weather.

The excavations and embankments shall be finished in a careful and workmanlike manner, true to line and grade as given by the engineer's finishing stakes. The bottom and side slopes must be made even and regular, fit for the passage of water, and any blasting in soft material which would injure the slopes or bottom will not be permitted. If the slopes or bottom are shattered or broken up by careless blasting, such soft places must be excavated to firm material and the holes thus made filled up to grade and line with concrete composed of one part of cement to three parts of sand and five parts of gravel or broken rock, all at the expense of the contractor. In hard material, where excavation is lined with concrete or plastered with mortar, all such holes must be filled up to the neat line of the back of lining, as specified, all excess over the specified thickness of lining being at the contractor's expense.

34.—Puddling.

Where the nature of the material in the banks is such as to require puddling by hand, the engineer will decide upon the depth of puddled material required on the face of the bank and stake out the foot of the bank at the proper distance from the edge of the excavation, to allow of the puddled material being placed afterward. The excavation in all cases to be made complete and the material required for puddling piled upon the bank ready for use, when a little water can be let into the canal.

35.—Tunnels.

There will be at least three tunnels on this work, one of 400, one of 900, and one of 1,400 feet in length, approximately. As the material encountered is variable, three methods of lining are proposed, as shown in the cross sections on drawing No. 1.

1. Where the material requires temporary timbering during excavation and permanent lining with concrete afterwards, the timbers remaining in the concrete.

2. Where the material is of such nature as to stand until lined, or to allow of the timbers being moved when the lining of concrete is put in.

3. When the material is hard rock, and can be plastered with a coat of cement plaster.

Any or all of these methods may be used in any one tunnel, or in all of the tunnels, as the varying nature of the material may require.

The object to be attained is the completion of a smoothly plastered chamber of concrete, 12 by 13 feet in section, in all the tunnels, to be made safe and secure against falls or "caves," and in order to effect this object, the use of any one of these methods may be insisted on by the engineer.

Any excess of excavation over the neat lines of excavation as shown on the drawings must be filled up with timber or concrete, as the engineer may direct. All caves or falls in the roof or sides must be supported by temporary timbering immediately after they occur, and the engineer will positively not allow the work to proceed until such places are made perfectly safe and secure. If more timber or concrete than is provided for in the drawings is required to make the work absolutely safe and secure, in the opinion of the engineer, such extra material shall be furnished by the contractor and allowed for by the engineer, at the rate of $30 per thousand feet, board measure, of lumber, and $12 per cubic yard of concrete, both measured *in* place; but no allowance shall be made for extra material used in the tunnel unless the engineer shall have previously given a written order for the use of the same. The frames, as shown in the drawings, are of 8 by 8 timber throughout, and set 3 feet apart from center to center. The lagging is of 2 by 6 inch timber. The concrete lining on sides and arch is 12 inches thick between timber frames, and 4 inches thick inside of frames. The concrete lining on the bottom is 4 inches thick. The concrete lining, where timbering is unnecessary, is 8 inches thick on an average, and the plastering on hard rock an average of 2 inches thick.

The contractor shall be fully responsible for the safety and good condition of the tunnels until the completion of his contract as a whole. None but skilled tunnel men shall be employed as foremen or carpenters, and of this the engineer must have satisfactory evidence beforehand. All losses resulting from damage or injury to person or property, through carelessness, neglect, or want of due precaution on the part of the contractor or any of his agents or employees, must be borne by him, and under no circumstances shall compensation for such losses be allowed to the contractor by the Secretary of the Interior.

36.—Explosives.

The use of explosives is positively prohibited in any of the tunnels, except where hard rock occurs, and in that case the blasting must be done in such a way as not to shatter the roof or sides unnecessarily.

37.—Centering.

The centering must be strong and solidly braced, and conform to the true outline of the tunnels as shown on the plans. It will be subject to the approval of the engineer, who may at any time prohibit the use of centering which, in his opinion, is not strong enough, or which may have deteriorated from use, or shown by actual trial that it is not fit for the purpose intended. The faces of the boards used on centering, for the sides of the tunnel, must be surfaced and laid true to line so as to give a plane, even surface of concrete for the plastering.

38.—Plastering.

When the concrete lining in any of the tunnels is completed, and the centering removed, the floor for the full width and the walls and arch for a height of 13 feet above the finished floor, shall be covered with a coat of plaster composed of sand, cement, and lime paste in the proportion of 3½ cubic feet of sand to 1 cubic foot of loose cement and half a cubic foot of lime paste, which has been cooled for at least twenty-four hours. The average thickness of this plaster will be half an inch, for which due allowance will be made in setting the forms for the concrete; and it must be troweled down on floor and sides to a perfectly smooth and true surface. The walls and floor must be thoroughly washed clean of all loose or friable material due to contact with centering boards, and then wet again to receive the plaster.

39.—Concrete.

The cement will be furnished by the Secretary of the Interior. The concrete to be used on all the structures and tunnels on this canal will be composed of Portland cement, sand, and gravel or broken stone, in the proportion of 1 barrel of cement, in the packed condition in which it is sold, to 7 full barrels of the same size of the aggregates when mixed together. To facilitate the work experiments will be made by the contractor with the carriers, whether wheelbarrows, boxes, or cars, used by him, so that this proportion of cement to aggregates may be maintained as nearly as possible, and the engineer will supervise these experiments and fix said proportion for the kind of carrier used at each piece of work. He will also make experiments with the aggregates themselves so as to get the most compact mass that can be made from them, and for such experiments no extra allowance will be made to the contractor. If the cement comes in sacks, then 380 pounds net of cement will constitute a barrel, and any ordinary cement barrel open at one end, containing not more than 3.7 cubic feet, will be used for measuring the aggregates. If broken stone is used, it must be hard and compact and satisfactory to the engineer. The entire product of the crusher will be taken, provided there is not more than 10 per cent of the volume composed of dust or screenings. All of the rock must be of such sizes as will pass through a screen with 2-inch square mesh. If gravel is used, it must be clean, hard, and heavy, having at least a specific gravity of 2, and screened into three different sizes. None of the gravel is to exceed 2 inches in diameter. The mixture of such sizes will be made in the proportion fixed by the engineer. A promiscuous mixture of sand and gravel will not be accepted. The sand must be clean and sharp and free from any clayey matter.

The mixing of the concrete, if hand labor is used, will be done in the following manner: A tight floor of either planks or sheet iron will be used for the mixing in all cases. The sand, which must be dry, will first be piled on the floor with the cement, in the proper proportions; the mass will then be shoveled over as many times as are necessary to make a thorough mixture of sand and cement; sufficient water will then be added to make a stiff mortar, and the mass shoveled over twice or more, as may be necessary. The stone or gravel, which should be well wet, will then be added, and the entire mass shoveled over twice or more before shoveling into the carriers. This mixing must be done to the satisfaction of the inspector. If the mixing is done by machine, the latter will be subject to approval by the engineer. If at any time the machine fails to perform the mixing in a manner satisfactory to the engineer, it must be made satisfactory or removed and another machine substituted, or mixing by hand be resorted to.

The water used for mixing must be clean and free from vegetable matter. The amount of water used, both in the mixing and in seasoning the concrete after it is placed in the work, must be satisfactory to the engineer.

The cement will be delivered to the contractor on board the cars at the nearest station to the work. The contractor will be responsible for demurrage to the railroad company, and will haul the cement from the depot to the work. He will furnish suitable warehouses or sheds for storing the same until used, and will be held responsible for any loss of cement after its delivery at the railroad station. The contractor must give the engineer at least thirty days' notice as to when and where he wants the cement delivered, and will state the amount required, which must be expressed in even carloads.

In all concrete walls over 2 feet thick hard bowlders or fragments of hard, sound rock not exceeding 1 foot or less than 6 inches in any dimension may be placed by hand in the soft concrete, provided no such stone comes nearer than 6 inches to the exterior surface of the wall or to any other bowlder or stone so placed. Under no circumstances is concrete to be laid in water or in a muddy foundation, and it must always be mixed and laid in the presence of an inspector and to his satisfaction. All concrete must be placed in the work as soon as possible after being mixed, and any portion of a batch which may remain in the place of mixing longer than the time required by the cement for initial set must not be remixed or placed in the work. All concrete shall be well tamped if put in dry, with heavy tamping bars, till moisture appears on the surface; and if wet, with suitable bars and shovels, so that porosity and rough surfaces may be avoided. Whether concrete will be used "wet" or "dry" will depend to a great extent on the nature of the cement, and this can be determined by experiment before work is commenced.

The wooden forms for concrete must be surfaced on the insides, so that all concrete structures may be smooth and true on their exposed surfaces when finished. If necessary, special flat shovels shall be furnished and used by the contractor to effect this purpose.

40.—Headworks.

The excavation and embankment involved in the temporary work necessary to divert the river during the construction of the headworks and the embankment for the permanent dam, will be paid for at the unit prices mentioned in the contract. The price bid on lumber for lining temporary channel will include all labor of laying and calking. As soon as the river is diverted, as shown on drawing No. 8, the excavation for the foundation of the headworks may proceed. A line of sheet piling, as shown on drawing No. 8, located about 2 feet from upper line of foundation, will be driven when the trench is excavated to the depth shown on said drawing. If it is found impossible to drive the sheet piling to the full depth required, it will be allowed to project into the concrete, but every effort shall be made to drive the piling so that not more than 1 foot of the piles may project into the concrete. No piling is to be driven deeper than this unless by written order of the engineer for extra piling, for which due allowance will be made to the contractor.

On the first layer of concrete, which shall be about 9 inches thick, in the foundation there shall be laid across stream seven lines, each 150 feet long, at equal intervals, of railroad iron, weighing not less than 50 pounds per yard, then a layer of concrete, which shall be well tamped around and level with the top of these rails. On the surface thus made shall be laid downstream 31 lines, each 30 feet long, of the same kind of rails, and then a layer of concrete around and level with the top of these rails. Between the rails will be laid, by hand, in the soft concrete, bowlders or large fragments of hard rock, as specified under the head of "concrete," to act as binders between the finished layers, and such bowlders or large fragments of hard rock will be required on the surface of each layer of concrete in the headworks, which is left to set before another layer is laid down. The layers of concrete should be not less than 9 inches nor more than 1 foot in depth.

On the downstream side of the foundation the old channel of the river will be excavated, if necessary, and filled with large rocks, paved on the surface, for a distance of 20 feet downstream and the full width of the river bed, as shown on drawing No. 7.

41.—Timber.

All timber must be of good quality, free from sap, shakes, loose or rotten knots, or other defects that would impair its strength or durability.

42.—Cast iron.

The cast iron should be of best quality of soft gray iron, free from cinders, blow-holes or other imperfections, and castings must be true to shape and dimensions, as shown on drawings. No plugging of holes will be allowed.

43.—Structures on line of canal.

All the structures on the line of the canal will be subject to these specifications, and where necessary special plans and specifications will be furnished by the engineer to the contractor.

44.—Highway bridge.

The bridge will consist of one 60-foot Pratt truss combination span with concrete abutments and of the dimensions shown on drawing No. 9.

The steel used in the bridge shall be open-hearth, with an ultimate tensile strength of at least 60,000 pounds per square inch. The chord links and main ties shall be die forged without welds, and of dimensions shown. The beam hangers shall be yoke shaped and bent around pin, one at each end of floor beam; pins to be turned to a perfect size and one-thirty-second of an inch less than the holes in chord bars and ties. The cast iron and timber should be in accordance with general specifications.

45.—Workmanship.

All workmanship shall be first class, and no bars of steel having an error of over one-sixty-fourth of an inch in length between pinholes shall be accepted.

46.—Chamber.

The bridge shall have a chamber of 2 inches in center, and the framing of the timber must be done by an experienced bridge carpenter.

47.—Paint.

All iron and steel is to have one good coat of metallic paint before leaving the factory, and the whole structure, except flooring and joists, is to have two good coats of paint composed of linseed oil and white lead.

The bridge must be built and completed according to plans and specifications, and to the entire satisfaction of the engineer.

Schedule of work for Truckee project, with amounts of bids on the several items.

DIVISION 1.

Character of work.	Amount.	Unit.	Mahoney Bros.	Atlantic, Gulf and Pacific Co.	Pacific Construction Co.	Chas. A. Warren & Co.	E. B. & A. L. Stone Co.
Excavation in water	2,000 cubic yards	Cubic yard	$1.00	$3.00	$5.00	$2.00	$3.00
Concrete	3,000 cubic yards	do	6.50	6.00	7.50	6.00	8.50
Sheet piling, 12 feet long, driven in place	200 linear feet	Linear foot	40.00	40.00	45.00	29.00	4.25
Gates and guides, complete, in place, cast iron	96,000 pounds	Pound	.08	.07	.07½	.07	.12
Pedestals and screwstems, complete, in place	24	Each	75.00	100.00	120.00	100.00	90.00
Flash boards, 3 by 12 inches, bound with ⅛ by 3-inch strap iron	104	do	2.00	1.00	1.00	1.00	2.00
Steel channels for pedestal support, with anchors	10,000 pounds	Pound	.06	.06	.06½	.07	.12
Lumber laid and calked in temporary channel of river	100,000 feet B. M	M. feet	40.00	40.00	42.00	50.00	50.00
Lumber for foot plank	5,000 feet B. M	do	30.00	35.00	36.00	40.00	40.00
Railroad iron for grillage in foundation	17 tons	Ton	60.00	60.00	58.00	60.00	50.00
Riprap	500 cubic yards	Cubic yard	1.00	2.00	1.50	4.00	1.50
Paving	4,000 square feet	Square foot	.20	.20	.15	.15	.15
Canal excavation and structure:							
Class 1, excavation	600,000 cubic yards	Cubic yard	.22	.35	.39	.23	.20
Class 2, excavation	60,000 cubic yards	do	.40	.70	.55	.80	.50
Class 3, excavation	60,000 cubic yards	do	.40	.70	.55	.40	.75
Class 4, excavation	60,000 cubic yards	do	.90	1.40	1.25	.80	1.00
Concrete	3,000 cubic yards	do	6.50	10.00	9.80	6.00	8.50
Gates and guides, complete, in place, cast iron	84,000 pounds	Pound	.08	.07	.07½	.07	.12
Pedestals and screwstems, complete, in place	21	Each	75.00	100.00	120.00	100.00	90.00
Flash boards as above	63	do	2.00	1.00	1.00	1.00	2.00
Steel channels for pedestal support, in place	9,000 pounds	Pound	.06	.06	.06½	.07	.12
Lumber for foot planks	5,000 feet B. M	M. feet	30.00	40.00	36.00	40.00	40.00
Puddling	1,000 cubic yards	Cubic yard	.50	2.50	3.00	2.00	1.50
Total for Division 1			309,369.00	468,102.00	475,846.00	308,417.00	350,214.00

DIVISION 2.

Character of work.	Amount.	Unit.	Atlantic, Gulf and Pacific Co.	Pacific Construction Co.	Chas. A. Warren & Co.	E. B. & A. L. Stone Co.
Excavation, class 1	500,000 cubic yards	Cubic yard	$0.35	$0.20	$0.23	$0.20
Excavation, class 2	40,000 cubic yards	do	.70	.55	.30	.50
Excavation, class 3	40,000 cubic yards	do	.70	.55	.40	.75
Excavation, class 4	30,000 cubic yards	do	1.40	1.25	.80	1.00
Concrete	30,000 cubic yards	do	a 10.00	a 9.80	5.50	8.50
Gates and guides complete, in place, cast iron	40,000 pounds	Pound	.07	.07½	.07	.12
Pedestals and screw stems, complete, in place	10	Each	100.00	120.00	100.00	90.00
Flash boards as above	30	do	1.00	1.00	1.00	2.00
Steel channels for pedestal support, in place	5,000 pounds	Pound	.06	.06½	.07	.12
Lumber for foot planks	3,000 feet B. M.	M. feet	35.00	36.00	40.00	40.00
Puddling	1,000 cubic yards	Cubic yard	2.50	3.00	2.00	1.50
Tunnels, 400 feet, 900 feet, and 1,400 feet, approximately:						
First method	1,500 linear feet	Linear foot	65.00	62.50	36.70	75.00
Second method	600 linear feet	do	58.00	64.00	37.20	55.00
Third method	600 linear feet	do	60.00	66.50	40.00	40.00
Total for Division 2			a 478,035	a 390,613	439,670	612,480

DIVISION 3.

Excavation, class 1	800,000 cubic yards	Cubic yard	$0.30	$0.24	$0.28	$0.20
Excavation, class 2	20,000 cubic yards	do	.70	.55	.40	.50
Excavation, class 3	20,000 cubic yards	do	.70	.55	.50	.75
Excavation, class 4	10,000 cubic yards	do	1.40	1.25	1.00	1.00
Highway bridge, complete, with concrete piers			1,500.00	1,600.00	1,500.00	2,250.00
Overhaul, 20,000 cubic yards, 500 feet	Cubic yards	100 feet	.05	.03½	.01	.02
Total for Division 3			288,500.00	231,600.00	258,700.00	199,250.00
Grand total			1,234,637.00	1,098,059.00	996,787.00	1,161,944.00

a Based on 3,000 cubic yards of concrete, which is the correct amount. Therefore, subtract 27,000 (unit price for concrete) from Warren & Co's. and from E. B. & A. L. Stone Company's total in order to make the bids comparable.

CONTRACT.[a]

THIS AGREEMENT, made and entered into this third day of September, one thousand nine hundred and three, by and between the United States of America, acting in this behalf by E. A. Hitchcock, Secretary of the Interior, party of the first part, and C. A. Warren and Company, a corporation duly organized and existing in the State of California, party of the second part, its successors and assigns;

WITNESSETH: That the said parties have covenanted and agreed, and by these presents do covenant and agree, to and with each other as follows:

ARTICLE 1. That the said C. A. Warren and Company shall construct and complete divisions 1 and 2 of the main canal and structures of the Truckee project in Nevada, division 1 being approximately 6 miles from the headworks of said canal, and division 2 being a section of 7 miles, approximately, in continuation of division 1, in accordance with the terms of the attached advertisement, proposal, and specifications, the same being made a part of this contract.

ARTICLE 2. The work herein provided for shall be commenced within thirty days from the date of the signing of this contract, and shall be completed, in full compliance with all the conditions and stipulations hereof, to the satisfaction of the Secretary of the Interior, within fourteen (14) months from said date.

ARTICLE 3. The contractor agrees to comply with the provisions of the acts of Congress approved August 1, 1892 (Stat. L., vol. 27, p. 340), and June 17, 1902 (Stat. L., vol. 32, p. 388), concerning the limitation of a day's labor to eight hours; and of the acts of February 26, 1885, and February 23, 1887 (Stat. L., vol. 23, p. 332, and p. 414) concerning the importation of foreigners or aliens under contract or agreement to perform labor in the United States.

ARTICLE 4. The contractors agree to save, keep, bear harmless, and fully indemnify the United States and any of its officers or agents from all damages or claims for damages, costs, or expenses in law or equity that may at any time arise or be set up for any infringement of the patent rights of any person or persons in consequence of use by the Department of the Interior, or by any of its officers or agents, of articles supplied under this proposal, and of which the contractor is not the patentee or assignee or lawfully entitled to sell the same.

ARTICLE 5. Should any question arise, at any time during the progress of the work, as to whether or not the contractor is properly carrying out the provisions of this contract in their true intent and meaning, or should the contractor fail to prosecute the work in such manner as to insure its completion within the time limit, notice thereof shall be served upon him, and upon his neglect or refusal to provide means for a more energetic and satisfactory prosecution of the work the Secretary of the Interior shall have the power to suspend the operation of the contract and employ other parties to complete the work at the contractor's expense, and any excess of cost arising therefrom over and above the contract price will be charged against the contractor and his sureties, who will be held liable therefor. If, by circumstances beyond the control of the contractor, and through no fault of his, he be prevented from completing the work within the specified time, such additional time may be allowed in writing, upon application therefor, as in the judgment of the Secretary of the Interior may be deemed just and reasonable.

ARTICLE 6. It is agreed that paragraph 27 of the specifications entitled "Unpaid claims," shall not apply to this contract, but in lieu thereof the contractor agrees to make promptly all payments to parties furnishing him with labor and material in the prosecution of the work, and that in pursuance of the act of Congress approved August 13, 1894 (Stat. L., vol. 28, p. 278), the bond executed in connection herewith shall contain a condition to that effect.

ARTICLE 7. If at any time during the continuance of the contract, the sureties or either of them shall die or become irresponsible, the Secretary of the Interior shall

[a] A general form of contract is given on pp. 110–112.

have the right to acquire additional and sufficient sureties, which the contractor shall furnish to the satisfaction of the Secretary of the Interior within ten days after notice, and in default thereof the contract may be annulled.

ARTICLE 8. That for and in consideration of the faithful performance of the stipulations of this contract, the party of the second part shall be paid by the disbursing officer of the reclamation service, on the presentation of proper receipts or vouchers in duplicate duly approved by the chief engineer upon the terms and conditions of this contract.

ARTICLE 9. That in case of the failure of the said party of the second part to comply with the stipulations of this contract according to the true intent and meaning thereof, then the party of the first part shall have the power to suspend said contract and complete the work at the contractor's expense, and any excess of cost arising therefrom over and above the contract price will be charged against the contractor and his sureties, who will be held liable therefor. The Secretary of the Interior alone shall determine what constitutes a failure or default hereunder, and such decision as he may reach in the matter will be final and conclusive.

ARTICLE 10. It is understood and agreed that the quantities of material and the classifications thereof in the specifications are approximate, and that payment shall be made for the actual quantities and classifications, whether more or less than those scheduled in the specifications. Any changes made in the details of the work shall be paid for at the unit prices of the proposal.

ARTICLE 11. The contractor agrees to furnish to the engineer such information as shall afford an accurate account of the number of men employed, their pay, and the time during which they worked on the various classes of construction.

ARTICLE 12. It is further understood and agreed that the corrections of typographical errors in paragraphs 8, 19, 28, and 36 of the specifications, as marked in the margins thereof, are accepted by both parties hereto.

ARTICLE 13. That it is expressly understood by the party of the second part that, in conformity to the requirements of section 3737 of the Revised Statutes, neither this contract nor any interest therein shall be transferred to any other party or parties, and that any such transfer shall cause the annulment of the contract so far as the United States are concerned; all rights of action, however, for any breach of this contract by the contracting parties being reserved to the United States.

ARTICLE 14. That it is further expressly agreed and understood that no member of or delegate to Congress, officer, agent, or employee of the Government is or shall be admitted to any share or part in this contract, or to any benefits which may arise therefrom, and the provisions of sections 3739, 3740, and 3741 of the Revised Statutes of the United States, relating to contracts, enter into and form a part of this agreement so far as the same may be applicable.

IN WITNESS WHEREOF, the undersigned have hereunto subscribed their names and affixed their seals the day and date first above written.

E. A. HITCHCOCK, [SEAL.]
Secretary of the Interior for and on behalf of the United States of America, Party of the First Part.

C. H. WARREN AND COMPANY,
By A. E. BUCKMAN, *Vice-President*,
A. J. RAISCH, *Secretary*, [SEAL.]
Party of the Second Part.

Two witnesses to each signature:

.....
P. O. address, ...
PAUL M. NEPPERT,
P. O. address, San Francisco.
A. HAMMOND,
P. O. address, San Francisco, Cal.

CEMENT FOR TRUCKEE CANAL.

In the foregoing contract the Secretary of the Interior agrees to furnish the necessary cement. To procure this, advertisements were duly inserted in the usual papers calling for bids to be opened on September 25, 1903. The advertisement and specifications are as follows:

ADVERTISEMENT FOR BIDS.

Sealed proposals, in duplicate, will be received at the office of the United States Geological Survey at Reno, Nev., until 2 o'clock p. m., September 25, 1903, and there publicly opened, for furnishing about 30,000 barrels of Portland cement for use in the construction of the Lower Truckee canal, and to be delivered f. o. b. at stations between Clarks, Nev., and Massey, Nev. Specifications and full information can be obtained from L. H. Taylor, resident engineer, Reno, Nev.

Each bid must be accompanied by a certified check, made payable to the Secretary of the Interior, for 3 per cent of the amount of the bid, as a guaranty that the bidder will enter into a written contract with the Secretary of the Interior within ten days after the award is made, and furnish bond in the sum of 50 per cent of the contract price for the faithful performance of the contract.

The Secretary of the Interior reserves the right to reject any and all bids.

Bidders are invited to be present at the opening of bids. Proposals should be addressed to L. H. Taylor, resident engineer, United States Geological Survey, Reno, Nev., and indorsed on envelope, "Proposals for furnishing cement."

CEMENT SPECIFICATIONS.

[Prepared by E. Duryee.]

1. *Kind of cement required.*—The cement shall be a Portland cement. By the term Portland cement is to be understood the material obtained by finely pulverizing clinker produced by burning to semifusion an intimate artificial mixture of finely ground calcareous and argillaceous materials, this mixture consisting approximately of three parts of lime carbonate to one part of silica, alumina, and iron.

2. *Bidders and brands.*—Tenders shall be received only from manufacturers or their authorized agents, and the name of the brand offered shall in all cases be stated in the bid.

3. *Weight.*—The average weight per barrel shall not be less than 375 pounds net. Four sacks shall contain 1 barrel of cement. If the weight, as determined by test weighings, is found to be below 375 pounds per barrel, the contractor may be required to supply, free of cost to the United States, an additional amount of cement equal to the shortage.

4. *Packages.*—If the cement is delivered in barrels, the barrels shall be strong and lined with paper, and the cement shall be free from lumps. Any package broken or containing damaged cement may be rejected by the United States agent in local charge.

5. *Samples for testing.*—Samples of cement are to be taken from the interior of the barrels or sacks with a suitable instrument. Samples are to be taken from every tenth barrel; they are to be numbered, and the packages from which they are taken are to be sealed and the numbers attached for future identification. The quantities taken are to be kept separate and tested separately.

6. *Aeration and testing.*—No cement shall be shipped until at least sixty days after its manufacture, and a longer period may be required by the Government engineer. The engineer shall give notice in writing to the contractor of the approximate amounts required for cement shipments and of dates for sampling. The contractor shall keep

in storage, in sacks or barrels, free of expense to the United States, for sampling and testing during a period of twenty-eight days, such stocks of cement as the engineer shall require.

7. *Factory inspection.*—The cement expert of the Government, or his authorized agent, shall at all times have liberty to inspect the materials, process of manufacture, and daily laboratory records of analyses and tests at the cement works. Whenever possible, unsatisfactory cement shall be rejected before shipment from the cement works.

8. *Chemical composition.*—The cement must be a Portland cement of regular and normal composition. It must contain at least 1.7 times as much lime as silica, alumina, and iron oxide together. The presence of over 3 per cent of magnesia, or 2 per cent of sulphuric anhydride, shall be considered sufficient ground for the rejection of the cement. Adulteration of the cement by the addition of any material to the cement after burning, with the exception of not over 2½ per cent of gypsum or plaster, will cause its rejection. The question of such adulteration may be determined either by chemical analyses or by inspection of the process at the factory.

9. *Fineness.*—Ninety-five per cent, by weight, must pass through a No. 100 sieve having 10,000 meshes per square inch, the wire to be No. 40 Stubb's wire gauge, and 75 per cent by weight must pass through a No. 200 sieve having 40,000 meshes per square inch, the wire to be No. 48 Stubb's wire gauge.

Specific gravity.—The specific gravity of the cement shall not be less than 3.

10. *Soundness.*—Pats are to be made of neat mortar of normal consistency, as determined by the Vicat needle. The pats are to be molded on glass plates. They are to be circular in shape, 3 inches in diameter, one-half inch thick in the center, and drawn to a thin edge at their circumferences. The pats are to be kept under a wet cloth until finally set. One pat is then to be put in water, which will be raised to the boiling point for six hours. If the pat softens, cracks, or disintegrates, the cement is unsound.

11. *Time of setting.*—The cement shall not acquire its initial set in less than forty-five minutes, and must acquire its final set within twelve hours. The pats made to test the soundness may be used to determine the time of setting. The cement is considered to have acquired its initial set when the pat will bear, without being appreciably indented, a wire one-twelfth inch in diameter loaded to weigh one-fourth pound, or when the 1-pound Gilmore needle fails to penetrate completely through the pat. The final set has been acquired when the pat will bear, without being appreciably indented, the 1-pound Gilmore needle.

Making briquettes.—In making briquettes, neat cement mortar of normal consistency, as determined by the Vicat needle, will be used. The mortar will be thoroughly mixed with a trowel and kneaded into molds with the thumbs, a blunt stick, or a plunger. Six briquettes will be made from each sample. In making sand briquettes the proportions shall be one part by weight of cement to three parts of standard crushed quartz sand and about half as much water as is used for neat briquettes. Six briquettes will be made from each sample.

12. *Tensile strength.*—The neat briquettes, prepared as specified above, shall stand a minimum tensile strain per square inch without breaking as follows:

	Pounds.
For one day in air and six days in water	450
For one day in air and twenty-seven days in water	550

The sand mortar briquettes, prepared as specified above, shall stand a minimum strain per square inch, without breaking, as follows:

	Pounds.
After one day in air and six days in water	175
After one day in air and twenty-seven days in water	225

The above are to be considered the minimum requirements. The tests of the neat cement briquettes are to be considered of less value than the tests of those made of

sand and cement. The twenty-eight-day tests must always be higher than the seven-day tests.

A cement that fails to meet any one of the above requirements may be rejected.

13. *Bags.*—If the cement is delivered in bags, the empty bags shall be returned to the contractor's agent at some railroad point convenient to the works of the Truckee project, and the sum of 10 cents shall be paid to the contractor for each bag not returned in serviceable condition, it being understood that all bags not torn nor with holes in them shall be considered serviceable.

INVESTIGATIONS IN PECOS VALLEY.

By W. M. Reed.

It has been publicly stated that irrigation in New Mexico is on the decline. This may be true for the Rio Grande Valley, but there is a section in the southeastern part of New Mexico, well known over the continent as the Pecos Valley, where the opposite condition exists. It is of this region that I wish to speak a few words.

This section has passed through the various stages of development common to many Western districts. First known as a portion of the great American desert, then as the home of the buffalo, next as the disputed territory and battleground of the Comanche and Apache, and later as the El Dorado of the cattleman, it is now the home of the advanced and prosperous American citizen. The Forty-niner, in his mad rush for gold, passed through one section of this valley, over the Butterfield trail, without even dreaming that here was to arise an empire rivaling in wealth the gold fields of the Farther West.

It was left for citizens of Utah first to discover the possibilities of its wealth and to begin its development by the construction of small canals and the production of crops by means of irrigation. The very beginning was marked by success, for although many minor failures were made, as is the case in almost every new country, the high prices paid for agricultural products by the cattlemen made it possible for the farmer to continue his experiments until he thoroughly understood the conditions and was able to obtain most wonderful yields and to reap large financial returns.

In a comparatively few years the normal summer flow of the rivers had been appropriated, and further progress, except by more intense cultivation, was practically at an end. About this time a leader came out of the wilderness, a keen, alert cattleman, who saw the possibilities of storage of the flood waters. A great irrigation system was planned, and most of the original projects have been finished. There are now, as a part of this system, two large reservoirs for impounding storm waters. One has a maximum height of 48 feet and a length of 1,380 feet, submerges an area of 1,980 acres, and has a storage capacity of 6,300 acre-feet. The other has a maximum height of 52 feet and a length of 1,686 feet, submerges an area of 8,331 acres, and has a storage capacity of 83,000 acre-feet.

These dams are of the rock-fill earth-apron type, and, barring an accident in 1893, due to lack of spillway facilities, have proved to be perfectly safe and satisfactory.

There are 60 miles of main canals and hundreds of miles of laterals supplied from these reservoirs. One of these canals crosses Pecos River. This crossing was made by means of a timber flume, but during the last year this was replaced by a modern arched concrete flume, strengthened and bonded by steel rails. Thousands of acres are now under cultivation. Fruit is shipped out not only by carloads, but by trainloads, and always commands the top market prices. Land that was once considered worthless is now producing abundantly, and areas that were mesquite and catclaw jungles are now veritable gardens.

Several hard problems have been met. Alkali and silt has given the usual scare, moss has choked the canals and laterals, and, worst of all, discouragement has sometimes attacked the farmers. All of these troubles have been met squarely and mostly overcome, and the future of this part of the Pecos Valley, Eddy County, dependent almost wholly upon the principle "Store the floods," looks bright and hopeful.

Farther up the valley, in Chaves County, with Roswell as a center, artificial storage has not yet been undertaken. Here three rivers formed by large springs have furnished most of the water used in irrigation, and no water from the Pecos River proper is used in this section. As these springs, which altogether have a flow of nearly 200 second-feet, discharge in a comparatively level country, the work of diverting and applying the water has been easy, and here some of the older and most improved places can be found. As yet there are no artificial reservoirs in this vicinity, but nature has provided one. More than one hundred wells, ranging in diameter from 2 to 8 inches, and varying in depth from 135 to 1,000 feet, have been bored down to this reservoir. The flow from these wells ranges from a few gallons to 1,500 gallons per minute, and no decrease has yet been perceptible. From these wells alone large tracts of land have been brought under cultivation, and the area is being increased.

The Pecos Valley is now passing through the driest season known there, yet, with the science and skill now acquired, it is standing the ordeal well, and is prosperous. Like other sections in the arid West, it is looking and hoping for Government aid in the construction of reservoirs for storing flood waters.

Since the passage of the reclamation act the engineers of the service have been doing considerable preliminary work in this section of New Mexico. Pecos River and its tributaries, from the point where the main stream enters Texas on the south to old Fort Sumner on the north, have been examined with considerable care and several projects have been surveyed in detail. Some of these have been temporarily passed by, not being considered feasible at the present time, but two,

known as the Hondo and Urton Lake projects, seem feasible, both from an engineering and financial standpoint.

Hondo River rises in the White Mountains and joins the Pecos at a point east of Roswell. The low-water flow has been appropriated and flood-water storage alone can increase the area under cultivation. About 12 miles west of Roswell is a reservoir site that presents very few engineering difficulties. There are no high or difficult dams to build, only light earthwork dams protected with riprap and inlet and outlet canals. The land under this project is very fertile, has all been filed upon, and is now in the hands of people who desire to cultivate it. It is estimated that by this project from 12,000 to 16,000 acres will be reclaimed, every acre of which will be farmed as soon as water is provided. The overflow population from the rich section about Roswell furnishes men who are ready to go to work at once, and the land will be as good as the adjacent land which now commands from $100 to $300 an acre. It is estimated that the initial cost for irrigating the land under this project will be about $20 an acre.

The area under the Urton Lake project is all Government land except a few claims bordering the river. This land is underlain by a very porous red sandstone, which will probably afford good subdrainage and prevent trouble from alkali. The reservoir site is a natural depression, well adapted to water storage, and will hold 190,000 acre-feet without any expense except for the outlet works, which will be of red sandstone.

The works contemplated in this project are: A dam and headworks on Pecos River, about 10 miles above Fort Sumner; about 36 miles of canal, to carry 1,500 second-feet in times of flood; three structures at crossings of creeks; the necessary outlet works, including a tunnel 7,000 feet long, and about 30 miles of distributing canals on the lands to be irrigated. A rough estimate of cost of this project is $1,020,050, or $17 per acre. There is little doubt that this land would all be settled on rapidly, as it is within 20 miles of the Pecos Valley and Northeastern Railroad, which line would undoubtedly build a branch to this section.

There are, however, some uncertainties in connection with the project which will have to be considered, one being the possibility that the reservoir will not be filled for some years. The data relating to stream measurements are not complete, and it will be necessary to continue the records of the run-off of Pecos River during the time the surveys of the canal and of the irrigable lands are being made and during the construction until the works are finished. It appears from the best information obtainable that the reservoir could be filled every ordinary year, but occasionally a dry year, such as 1903, creates a doubt as to the possibility of always filling the reservoir. It is probable that the seepage from the irrigated lands considered under these

projects would eventually more than make up for any apparent loss below them occasioned by the proposed disposal of the water from the Pecos River, and that no difficulties would arise with the Pecos Irrigation Company, which holds some water rights 150 miles farther south.

Investigations are now being carried forward on both of the last-named projects and a later report will give more definite and detailed information.

EXAMINATIONS IN NORTH DAKOTA.

By CHARLES H. FITCH.

An effort was made early in the present field season to locate a possible project for the reclamation of land in the State of North Dakota. To Mr. F. E. Weymouth, assistant engineer, assisted by Mr. John N. Kerr, assistant engineer, was assigned the duty of making a reconnaissance of the entire western half of the State. In the course of his investigations Mr. Weymouth visited every important stream in this region, making measurements of many of them, following watercourse after watercourse, and finally determined, with one exception, that there was no possible reclamation project which appeared to be in any way feasible.

This single exception is a small body of land close to the western boundary of North Dakota, near the mouth of Yellowstone River. There is very little irrigation from the Yellowstone along its course in Montana. It rises in the Yellowstone National Park, in Wyoming, its source being in Yellowstone Lake, from which it flows in a general northerly direction. Entering Montana it flows eastward nearly to the eastern edge of the State, when it turns abruptly, flowing a little east of north. At the turn of the river the Northern Pacific Railroad leaves it, and it empties into the Missouri just beyond the line of North Dakota, near the Great Northern Railroad. The distance between the railroads by river is about 90 miles.

It was decided to investigate the possibility of diverting the Yellowstone, and Mr. Weymouth organized a field party for this purpose. As this project would, if feasible, irrigate lands in Montana as well, it was desirable to cover as much land as possible without regard to State boundaries, particularly as the water supply is undoubtedly abundant for all lands that might come under it. The first line for diversion was started on the left bank of the river about 40 miles above Glendive, Mont. After running to Glendive it was found that this line failed to reach the bench land near Glendive, and that the line would be not only very expensive but that it could not reach any considerable body of land, the river here being very narrow and hemmed in by rough and broken hills.

Another attempt was then made about 1½ miles above Glendive,

also starting on the left bank. This line will undoubtedly cover the lower valley of the Yellowstone, which, about 45 miles below the diverting point, widens considerably, embracing from the bench land to the river a strip varying from 1 mile to about 5 miles in width. This means that about 45 miles of canal must be constructed before covering any land, and that the amount which can be irrigated is still unknown. The North Dakota land under this project will all probably be bottom land or valley land next to the river, and, estimating from the State map, will amount to about 20,000 acres.

IRRIGATION IN NORTH DAKOTA BY PUMPING.

By F. A. Wilder.

The purpose of this paper is to state briefly some of the conclusions reached as a result of work carried on during two summers in the western part of North Dakota, undertaken jointly for the State geological survey and the United States Geological Survey.

Work was begun during the summer of 1902, with the assistance of Mr. L. H. Wood, of the University of Chicago, and during the last summer was continued by two parties that operated along the Missouri and its tributaries in North Dakota.

The average rainfall in the western half of the State is about 18 inches, and is very often so distributed through the growing months as to yield, in connection with the naturally fertile soil, magnificent harvests. Farming conditions, however, are rendered somewhat uncertain by a fluctuation in rainfall.

The Great Plains of North America, and at least portions of the Mississippi Valley, seem to pass through cycles, or periods of years, of abundant rainfall, which alternate with periods when rainfall is more restricted. At present, and for three years past, taking the area as a whole, there has been an abundance of moisture, and for some States an oversupply. The sloughs of Iowa, which many predicted would never be filled again, are overflowing, and floods have occurred during spring, summer, and fall. During this period of excess, western North Dakota has enjoyed rainfall sufficient to bring to harvest abundant crops. When the sloughs of Iowa dry out again, as they surely will, those who farm in western North Dakota must irrigate or be content with limited returns.

With even a small fraction of the western part of the State under irrigation, the productiveness of the whole region will be immensely increased. Every river valley contains some land that may be artificially watered. With a few acres under perfect control, and abundant range for cattle in the rolling or broken land back of the valley, 10 men, combining farming with the raising of cattle, will prosper where one finds a living now.

During the last two summers over 200,000 acres along the Missouri and its tributaries, the Little Missouri, Heart, Knife, Cannon Ball, Yellowstone, and smaller streams flowing in from the north and east and standing 20, 40, 60, and 80 feet above water level, have been studied. Five or six large tracts, where from 5,000 to 15,000 acres can be controlled by a single enterprise, have been reported upon as possible for development. Most of the land, however, is in tracts of from 100 to 2,000 acres, and its development will depend upon private enterprise.

Perhaps 7,000 acres in the western part of the State are irrigated to some extent at present, operations being most active in Williams County. Coulees and creeks which empty into the larger rivers have been dammed, and their water has been diverted to the creek benches and river flats. Operations are generally confined to spring flooding. Heavy crops of oats and hay are taken from land saturated with water by flooding in the spring, and there is no record of failure where this plan has been followed, though in some localities operations extend back to dry years when the nonflooded land about was barren. Gumbo flats treated to water in this way have changed their vegetation from cactus and sage brush to a rich growth of blue-joint grass, which makes the best of native hay and compares well with timothy in value as fodder.

Irrigation by means of reservoirs in smaller tributaries which render it possible to water the flats of the larger streams, can be carried much further than it is at present, and is to be recommended wherever practicable. The difficulty which the plan presents is found in putting in dams of sufficient strength without too great an outlay of money. On account of the low gradient of the larger streams, the common practice of diverting water for irrigation by lateral ditches will prove practical only in rare instances in North Dakota. The presence of vast quantities of lignite, however, along all of the streams in the western part of the State, renders worthy of consideration the plan of pumping water from the streams up to the flats. The lift of course, must be moderate, but many of the flats do not lie high above water level in the streams.

Lignite is very abundant along the Missouri as far south as Bismarck, and below this city is found at a number of points, though the beds are generally thin. Outcrops occur every few miles along the river above Bismarck, and prospecting will doubtless reveal beds not now exposed. On the Missouri over sixty exposures more than 4 feet thick were measured during the past summer, many of the beds reaching a thickness of 12 or 15 feet, and showing for long distances.

On the Little Missouri and its tributaries, coal is nearly continuous in outcrop and reaches a maximum thickness of 40 feet on Sand Creek,

a tributary southwest of Yule post-office. Similar conditions hold for the Heart, Knife, Upper Cannon Ball, and Cedar rivers. Lignite is scarce on the lower Cannon Ball and thick beds are nearly absent for 100 miles from its mouth.

In but two instances is water now pumped for irrigation in North Dakota, lignite furnishing the motive power. These plants are on too small a scale to demonstrate the highest possible value of the method, though their proprietors are well satisfied with the profits obtained. On Green River, north of Dickinson, Fischer Brothers irrigate about 80 acres annually, the crops raised being oats, millet, potatoes, and garden products. A farm engine drives a rotary pump which raises 1,500 gallons a minute 15 feet, an elevation sufficient to distribute water over large areas on Green River and similar streams in the State. The cost of the pump was $250, delivered. Mr. Fischer estimated that his equipment irrigates 200 acres even in a dry year. Two dollars' worth of coal was burned each day, lignite costing 50 cents a ton at a neighboring bank. Mr. A. F. Riley, on Little Heart River, near Gladstone, in the summer of 1902 was preparing to irrigate 60 acres in the same way, using lignite from his own bank.

On the smaller streams that do not flow constantly temporary dams can be thrown across the stream bed, and these will retain water for irrigating the adjacent flats. The pump and engine can be moved from place to place, and the cost of ditching be thus greatly reduced.

Estimates are now being made as to the cost of lifting water sufficient even in dry years to irrigate these river flats with elevations taken at 20, 40, 60, and 80 feet, and lignite costing from 50 cents to $2 a ton. The results are not, however, ready for publication.

Away from the streams it may be practicable to use lignite for pumping from wells, for water is abundant at moderate depths throughout the lignite area. Where the area to be irrigated is small, not exceeding 100 acres, and cheap assistants to care for an engine are not available, it may be practicable to pump with gasoline, which may be a cheaper fuel than lignite. Two landowners in Williams County are now irrigating in this manner, and are fully satisfied with the results.

CONDITIONS IN SOUTH DAKOTA.

By Charles H. Fitch.

Work by the reclamation service in western South Dakota was commenced in May last by Mr. R. F. Walter, engineer, who was instructed to make a general reconnaissance, particularly in the Black Hills and the country adjacent, and to establish gaging stations on all of the important streams.

The Black Hills, noted especially as a mining region, are situated at the western edge of the State, and are encircled by two streams that head near together in Wyoming and flow in opposite directions, one

north and the other south. The first is the north fork of Cheyenne River, or Belle Fourche River; the second is the south fork of the Cheyenne. These streams, after completely circling the hills, join nearly east of Deadwood, 60 or 70 miles away, forming Cheyenne River, a tributary of the Missouri. All of the streams flowing out of the Hills are tributaries of one of the forks of the Cheyenne.

On the south fork of the Cheyenne it is possible that a storage reservoir may be constructed, but the project involves a high dam across the river, and it is doubtful if enough land could be reclaimed to warrant the expense; yet the scheme deserves further investigation. North of the point considered there are sites for a number of lesser projects which may be developed from smaller streams. One of these involves the storage of water on Box Elder Creek, including the water of that stream and Rapid Creek as well, though it is not positively known that Rapid Creek can be diverted as suggested. The Belle Fourche is a flood-flow stream—that is, it carries ordinarily about 100 second-feet, which in extreme low water drops to 50 second-feet, but it always has considerable flow after the melting of snow in the spring, and several floods usually occur during the summer months, when the flow sometimes amounts to 5,000 second-feet for several days at a time. During the present season there have been an unusual number of floods. Just at the town of Belle Fourche, Red Water Creek empties into the Belle Fourche. It is also a flood-water stream, but receives from Spearfish Creek a regular flow of considerable amount. Some of this stream is diverted and used for irrigation, but about 100 second-feet flows continually into the Belle Fourche.

As a large body of fine land lies on the north side of the Belle Fourche, and there are several sites for storage at a distance from the river, a feasible project seemed here to present itself.

The details of the survey now in progress and the problems that have arisen will be stated by Mr. Walter, the engineer in charge.

SOUTH DAKOTA INVESTIGATIONS.

By RAYMOND F. WALTER.

Work was begun in South Dakota in May, 1903, by a preliminary examination of the situation. Various streams were examined, reservoirs thought suitable for survey were reported upon, and gage rods were set and rating stations established on each stream, so that the available flow and length of the flood period, as well as the monthly run-off, could be determined. The principal streams studied were as follows:

Cheyenne River, from the Wyoming line to a point 30 miles below the mouth of Cascade Creek, with a gaging station at Edgemont, S. Dak. A large reservoir site was found below the mouth of Cas-

cade Creek, from which a ditch 35 miles long would water about 35,000 acres of private land known as the W. G. flats.

Beaver Creek at Buffalo Gap. The summer flow of Beaver Creek is all used for irrigation, and the unused flow is not considered large enough for extensive reclamation, although if stored it would probably help out largely for lands now farmed which are short of water.

French Creek. This stream has little or no available flow developed so far, and the lands under it are poor and rough.

Battle Creek. There is a gaging station established at Hermosa, on Battle Creek. Along this creek there are some fair reservoir sites, with some good land.

Spring Creek. About 9 miles south of Rapid City there is a site for a reservoir on Spring Creek, with some good lands under it, but the supply is doubtful unless, upon further investigation, it is found feasible to prevent loss from the bed of the creek as it flows through the foothills. It is estimated that this loss amounts to from 50 to 100 per cent in about 4 miles.

Box Elder and Elk creeks. These streams furnish reservoir sites north of Rapid City, where water, together with the available flow from Rapid Creek, might be stored to irrigate 30,000 to 50,000 acres of first-class land situated on the divides between Elk Creek and Rapid Creek. This is known as the Box Elder project, as the reservoir selected is situated on Box Elder Creek. This project will be investigated and the available water supply determined.

The Belle Fourche project, on which surveys are now in progress, is situated at the northern extremity of the Black Hills, the water supply being from Belle Fourche River and its main tributary, Red Water Creek. The project is located in Butte County, and its headworks are at the town of Belle Fourche, below the mouth of Red Water Creek. The flow of Belle Fourche River ranges from 50 cubic feet per second during low-water stage to 5,000 cubic feet per second during flood season. The drainage area is large, as it circles the western and northern part of the Black Hills, heading in Wyoming, and as the flow is precipitous and the soil rather impervious, the run-off is large and floods occur at any time during the summer months, caused by rain storms in the hills. The Red Water has a steady supply, fluctuating but little during the year. In this drainage area there is but one important ditch, which is diverted from Red Water Creek and irrigates 6,000 or 8,000 acres lying on the east side of the Red Water and the south side of the Belle Fourche. There are, however, a number of small private ditches from Spearfish Creek, a branch of Red Water Creek, but as the lands they irrigate all lie directly adjacent to the streams, much of the water is available as return seepage on lower Spearfish and Red Water above the head of the proposed canal.

The first reservoir contemplated under this project is on Dry Creek and is to be formed by an embankment about 3,600 feet long, to be of earth riprapped with brick, if it is found by further investigation and tests that bricks can be burned of sufficient hardness from the material at hand at a sufficiently low cost. All permanent works on canal and reservoirs are to be of concrete, as unlimited quantities of shattered rock for this purpose are obtainable by a short haul. The inlet to this reservoir will be 8 miles long, and is surveyed, located, and staked, as well as the outlet canal and the extension of the main line, for 70 miles, covering about 130,000 acres, nearly 80 per cent of which is vacant public land. The percentage of irrigable lands can not be determined until topographic surveys have been completed and the lands classified.

The outlet from the Dry Creek reservoir is located and staked for about 30 miles, covering all but about 15,000 acres of the above-mentioned lands, and when estimates are made it may be found to be cheaper to construct on this line for the main canal than on the upper line, which is much rougher and longer, and the cost of irrigation of lands between them may be found to be comparatively too great.

One other site has been selected and is being surveyed. This, known as the Wilson reservoir, is located 18 or 20 miles east of the Dry Creek reservoir, and is intended to furnish a water supply for the lower lands. The upper reservoir will cover about 1,500 acres at high-water contour, with greatest depth of 45 feet. It will be possible to convey water across the river from this reservoir, where about 15,000 acres of first-class land on the south side of the Belle Fourche can be irrigated.

The general elevation of the lands to be irrigated is about 3,000 feet above sea level. The soil ranges from a heavy clay to a rich black loam, but tends to become sticky "gumbo" when wet, and will be best adapted to hay, grain, and some fruits, although sugar beets, it is believed, can be grown profitably. The water in the river carries considerable silt during floods, which, if deposited in the reservoirs, would require further investigation as to the method of clearing them.

INVESTIGATIONS IN OREGON.

By JOHN T. WHISTLER.

There is a general impression, particularly in the East, that Oregon is a land where the sun rarely shines and where the people are "web-footed." For the purpose of correcting this wrong impression it is necessary to call attention to a few facts concerning Oregon before speaking of its future irrigation possibilities and of what the reclamation service is doing in the State.

The Cascade Mountains divide the State into two parts, which prob-

ably have a greater diversity of interests and climate than can be found in any other State in the Union, not excepting California—at least the line is more clearly drawn. Over two-thirds of the State, an area nearly as large as the State of Washington, the average annual rainfall is estimated to be less than 12 inches. Over the western third of the State the rainfall is certainly five times as great.

It is from the climatic conditions prevailing in this western part of the State that the popular impression of the State has been derived. Leaving out of consideration certain valleys in the Blue Mountains, we may say that there is as much difference between these two parts of Oregon as there is between northern California and Nevada; in fact, the central third of the State is a continuation of the Great American Desert, where routes of travel are determined by the few watering places.

The famous wheat lands of northeastern Oregon have but about 15 inches of rainfall, and 50 to 60 bushels of wheat to the acre are often raised where the land has to lie fallow for one year in order to obtain moisture enough for one crop.

To nine Easterners out of ten the idea of building irrigation works in Oregon, the home of the "webfooter," is like carrying coals to Newcastle, and yet two-thirds of the State averages less than a foot of rainfall in the year.

The condition of eastern Oregon, and more especially central Oregon, can in no way be better impressed on one's mind than by the knowledge of the fact that the point in the United States most remote from a railroad is in central Oregon. Three-fourths of this part of the State, or nearly half the entire State, is still vacant land, subject to entry.

Of a population of less than half a million, over 80 per cent is west of the Cascades, and a majority of the people in eastern Oregon have come, in the last decade, from "the valley"—meaning Willamette Valley.

About one-quarter of the area of the State is occupied by the Blue Mountains and the surrounding valleys. It is in this region and along the eastern foot of the Cascades that the possibilities for irrigation lie.

The Deschutes River drains the northern two-thirds of the eastern slope of the Cascades, and the Klamath River the southern third. The approximate run-off east of the Cascades is, roughly, 12,000,000 acre-feet, which is about 30 per cent of the estimated precipitation. This is sufficient to irrigate possibly 3,000,000 to 4,000,000 acres of land, and represents the extreme limit of irrigation in the State, assuming that the entire run-off could be utilized. From the present limited investigation it seems improbable that more than 50 per cent of this can ever be realized.

The immense summer range for stock which lies back of all irrigation possibilities, and which will unquestionably be increased by proper regulation of the forest reserves and future legislation regulating grazing on public lands, precludes any possibility that the supply of alfalfa will ever equal the demand.

The reclamation service is making surveys and other investigations, preparatory to preliminary and comparative estimates at the close of the season, on three projects, viz:

(1) The Umatilla project involves diversion and storage of about 200,000 acre-feet of water from Umatilla River and the reclamation of not less than 50,000 acres of land, which raises 40 to 60 bushels of wheat to the acre where it lies high enough to receive 15 inches of rain during the year. Looking forward to this project years before the reclamation act was suggested, the division of hydrography of the United States Geological Survey has had measurements of the discharge of Umatilla River made, covering a period of eight years. The discharge varies from 300,000 to 600,000 acre-feet per year. A weak point in the project is the lack of natural storage basins.

(2) The Malheur project contemplates the storage of the spring floods of Malheur River and the diversion of this water onto about 40,000 or 50,000 acres of land lying just across the Snake River from the famous Weiser and Payette tracts in Idaho. One of the best examples of irrigation now to be seen in the State is the famous "K. S. & D." ranch, the lands of which are owned largely by Mr. Fred Kiesel, of Ogden, chairman of the executive committee of this congress. Measurements made of Malheur River by the United States Geological Survey covering parts of five years indicate a discharge of from 200,000 to 400,000 acre-feet per annum. Good reservoir sites are found for this project. Long and costly canal construction is the drawback to the enterprise.

(3) The Harney project consists of storing the spring flood water of Silvies River and diverting in onto lands in Harney Valley, a beautiful valley comprising over 600 square miles, lying at an altitude of over 4,000 feet. The high altitude and the long distance from transportation facilities are, of course, features of the project that must be considered. Notwithstanding the usual Fourth of July snowstorm, two crops of alfalfa can be matured, and some excellent gardens and orchards of hardy fruits are now seen. Measurements for determining the amount of water available are being now carried on, and will be continued until construction is either decided upon or definitely abandoned.

The reclamation service is practically barred from investigation looking to construction in Deschutes River basin, owing to three Carey act projects, one of which has been approved by the Department; and the Klamath region has not yet been investigated.

The feeling is very strong in eastern Oregon that that portion of the reclamation fund derived from the sale of public lands in the State should be spent in the State, regardless of beneficial results. It is a very common belief that under the law it can not be spent elsewhere. It is remembered that it was in deference to the wishes of the late Congressman Tongue, of this State, that the clause which provides that the major portion of the funds derived from the sale of lands in any State should be spent in that State in ten years was inserted in the act.

The laws of the State regarding water rights are very primitive, and the earlier court decisions, given in cases arising in the Willamette Valley, to the effect, practically, that the less water a man has the better off he is, have established precedents which will make it difficult to adjudicate water rights in the arid portion of the State, where beneficial use must ultimately determine the right to control. It is in the Oregon statutes that one finds that the notice of an appropriation of water must state among other things how many "cubic inches of water, measured as miner's inches under 6 inches pressure," etc., it is desired to appropriate, and this has stood on the statutes since 1891.

The last legislature provided for the appointment of an irrigation commission, to consist of two attorneys, two practical irrigators, and one civil engineer, familiar with irrigation projects of the State, to report a bill to the next legislature, convening in 1905, repealing present laws and enacting new ones, but provision was not made for defraying the expenses of the commission.

INVESTIGATIONS IN UTAH.

By GEORGE L. SWENDSEN.

BEAR RIVER.

Though Bear River is one of the largest and most important streams in the Great Basin drainage area, nothing has yet been done to effect the storage of its flood waters. And each year, from the end of the irrigation season in September to the beginning in May, and even after the opening of the season, a very large volume, sometimes as great as 5,000 second-feet, of the water of the Bear goes to make up the amount annually evaporated from Great Salt Lake.

The Bear has its source in the Uinta Forest Reserve, about 60 miles southeast of Ogden. The height of the range of mountains forming the watershed may be made apparent by mentioning the altitudes: Bald Mountain, 12,250 feet; Mount Agassiz, 13,000 feet; Mount Wilson, 13,000 feet, and Burro, 12,800 feet, scattered in an east and west line along the summit. The river in its winding course passes first

through a section of Wyoming, flowing nearly due north, touches the borders of Utah for a short distance, and turns northwestward into Idaho, reaching the northernmost part of its course in the vicinity of Soda Springs, whence it turns southwestward and discharges into Great Salt Lake, about 12 miles northwest of Ogden. Among the important streams that flow into the Bear in its course are Rock Creek and Smiths Fork, in Wyoming; Thomas Fork, Montpelier, Soda, Cottonwood, and Mink creeks, in Idaho; Cub and Logan rivers, in Utah.

Not far from the southeastern border of Idaho, Bear River enters the north end of Bear Lake Valley, a long, narrow valley lying at an elevation of about 6,000 feet and covered, in the main, by Bear Lake. At its entrance to Bear Lake Valley the river is considerably above the level of Bear Lake; 15 miles farther down its course it is considerably below the lake level.

Bear Lake is very deep, its area is a little more than 100 square miles, and its shores for the most part are precipitous, so that a change of level of the water in the lake within reasonable limits will have but little effect upon surrounding lands.

The difficulties that may be encountered in the construction of channels from the river to the lake are scarcely worthy of mention.

At the present stage of the investigation it is impossible to state definitely the volume of unappropriated water available in Bear River at its entrance to Bear Lake Valley, but the measurements of the last year indicate that it will approximate 200,000 acre-feet a year.

By the combined flow of the small streams the volume of water available for storage in the lake would be increased considerably beyond this amount, perhaps to 250,000 acre-feet. The possibility of more fully utilizing Bear River for irrigation has been long understood by a number of prominent citizens of southern Idaho and northern Utah, but I know of no plan that promises general and complete results. With such a plan in view the examinations made are proceeding on a broad basis, no point of probable interest being left out of the investigation. The work done indicates that there will be greater engineering difficulties encountered in constructing the canal systems necessary to properly utilize the river than will be met in the work of converting Bear Lake into a reservoir to store the water, and a great amount of investigation will be necessary to determine the most feasible plan. The areas that may be benefited by the scheme are varied and extensive. The elevation of the river with relation to some of the large areas of vacant land in southeastern Idaho is such as to make it apparently the natural source of supply of the water needed to develop these sections, and already, by private enterprise, it is supplying portions of the vast area in the vicinity of Bancroft, Idaho. In that district a very large area of arid land can be reclaimed from

Bear River. The drainage from this area would go into Portneuf Creek, a small stream of the Snake River drainage basin. This drainage can be intercepted and utilized on vacant lands in the western part of Bannock County, along the Oregon Short Line Railroad, and it is probable that the canal system, when fully extended, will reach into the southwestern part of the county, and return a portion of the water to the Bear River drainage.

As Bear River approaches the northern boundary of Utah further opportunities are afforded for its utilization. By preliminary investigations it has been found that the shortage of water in the northern part of Cache Valley in Utah may be supplied from the Bear to increase the possibilities of irrigation in that very fertile district. Partially developed private enterprises also indicate that much of the land in the southern part of Oneida County, Idaho, and the western part of Cache, and the eastern part of Boxelder counties, in Utah, now irrigated in part, may be reached by canals from the Bear. As we reach the Utah section questions become much more complicated and will require careful consideration. The drainage from Cache Valley, one of the oldest irrigated sections, forms an important factor, and storage on the more important streams of that section will necessarily receive attention. The flood waters of that valley are entirely unappropriated, and definite information relative to their volume is already available in the records of the United States Geological Survey. The volume represented in the vested rights is also well determined and this item will need careful consideration.

The vested rights of the canals already taking water from the Bear in this section and those of the canals now in process of development are also of vital importance. However, a thorough investigation of the question from all points of view will certainly reveal a plan whereby the great volume of water which passes all the head gates of canals and reaches Salt Lake may be utilized. Though we may store all the unappropriated waters of the Bear that can be taken into Bear Lake, the Logan, with a flood discharge of more than 1,000 second-feet, the Cub with 500, Mink Creek with 400, the smaller creeks with an aggregate of more than 300, will all be lost in the Great Salt Lake. Therefore, while the use of Bear Lake is a leading feature in the utilization of Bear River, that is by no means the only point at which the question of storage should be considered.

I am fully aware that a complete knowledge of the situation will involve the investigation of the river above Bear Lake Valley and even to the headwaters of the streams, but the natural point at which to begin seemed to be Bear Lake, and we are working from that point toward the mouth of the stream.

In conclusion I shall venture one more suggestion in connection with Bear River and its relation to the plan for a more complete supply of

water for the populous part of Utah. Already a canal is partly built (though once practically abandoned, it recently obtained new life) to carry water from Bear River into Ogden. I am not familiar with the details of the plan and its contemplated scope, but in attempting to determine the best use of Bear River and at the same time keeping in mind the great necessity for more water in Weber, Davis, and Salt Lake counties, it has seemed to me that Bear River should be considered in connection with the latter question. Certainly natural conditions are such that engineering difficulties do not stand in the way of a combination of the Bear, Weber, and Ogden rivers irrigation interests. I have no detailed information in the matter, but my general information leads me to this conclusion. And though there may be difficulties which prohibit such a combination or exchange of interests as this, it seems to me that the feasibility of using Bear River water in the portion of Utah referred to may be properly considered in connection with the Bear River enterprise under the Federal reclamation law.

UTAH LAKE.

Utah Lake is a body of water lying in the north-central portion of Utah, and covering an area of about 140 square miles. It occupies a prominent place in the irrigation systems of the most populous part of the State, and has afforded a field for extended investigations in almost every line relating to irrigation. The main drainage to the lake is from the mountains lying east and southwest of the lake valley, the principal streams being Provo and Spanish Fork rivers, comparatively small streams, though with watersheds of considerable elevation and extent.

The only outlet from the lake is through Jordan River, which drains into Great Salt Lake. All streams entering the lake have been developed for irrigation to the full capacity of the flow during the irrigation season, yet much of the area apparently dependent upon this source of supply is but scantily supplied with water. This condition has suggested the idea of providing storage for flood waters and otherwise increasing the available water supply. The matter was brought to the attention of the reclamation service by the Utah arid land commission, provided for by action of the State legislature of 1902, and it was urged by this commission that this be made the Federal project to receive first attention in Utah.

Investigations were begun early in May of this year and the entire situation is being carefully examined. The plan of investigation involves a thorough and complete study of the present use of water in the district, as well as a determination of the possibility of storing and utilizing the flood waters.

The investigations have not progressed far enough to draw general

conclusions, and the problem will necessitate extended and thorough examination of the watershed, the streams, the present canals, the lands, and all other features which could in any way affect the project.

The element of evaporation is important, and the inquiry will consider the possibility of partially checking this serious loss. This may involve a reduction of the area of the lake by some means, or the storage of the water in the mountains, the lake being eliminated from the system. Examinations of the lake basin show that it is very shallow, and that there is no natural outflow during the summer months.

The following table gives a summary of the conditions of inflow to Utah Lake and losses by evaporation from the lake. It also shows the rise of the water due to the excess of inflow over evaporation and the fall (indicated by a minus sign) in the months from April to October, inclusive. The last column of the table shows the amount of water utilized. By comparing the column headed "Evaporation" with that headed "Utilized" it will be seen that only a small proportion of the water of Utah Lake is put to beneficial use. The evaporation is estimated at over 420,000 acre-feet and the amount utilized at less than 86,000 acre-feet; that is, less than 28 per cent of the amount evaporated is put to beneficial use.

Summary of conditions at Utah Lake.

Month.	Inflow.		Evaporation.		Rise or fall.		Utilized.	
	Second-feet.	Acre-feet.	Second-feet.	Acre-feet.	Second-feet.	Acre-feet.	Second-feet.	Acre-feet.
January	207	12,834	123	7,626	434	26,908	.....	.....
February	247	13,832	25	1,395	538	30,132	.....	.....
March	307	19,034	228	14,136	622	38,595	.....	.....
April	343	20,580	587	35,247	−51	−3,100	117	7,011
May	477	29,562	792	49,104	−161	−10,010	257	15,916
June	550	33,000	1,136	68,169	−604	−36,255	272	16,333
July	.....	.....	1,176	72,912	−1,071	−66,387	206	13,452
August	.....	.....	955	59,241	−817	−50,652	200	12,184
September	9	540	1,011	60,636	−581	−34,875	162	[a]9,968
October	116	7,192	501	31,062	−308	−19,065	187	11,149
November	200	12,000	208	12,462	252	15,112	15	921
December	200	12,400	138	8,556	375	25,250	.....	.....
Total	255	160,974	576	420,546	.....	.....	236	86,936

[a] The utilized result stated in this case is probably too low.

WATER LAWS OF UTAH.

By F. S. Richards.

At the last session of the Utah legislature a board of commissioners, consisting of five members, known as the Arid Land Reclamation Fund Commission, was created, and in March, 1903, the governor appointed Abraham F. Doremus, Joseph A. West, Hardin Bennion, William E. White, and L. R. Anderson members of said board. The law makes it the duty of the commission to take such measures as may be necessary to secure the construction by the United States Govern-

ment of such reservoirs and irrigation works in this State as are contemplated by the national irrigation law, and to insure the expenditure therefor of such proportion of the reclamation fund provided by said act as should properly be expended in this State; and the commission is authorized to employ assistants and incur such expense as may be necessary to enable it to carry out the purposes of the State law, an appropriation of $6,000 being made for this purpose.

Soon after the commissioners were appointed, the board met and considered the various irrigation and reservoir propositions within the State which had been suggested as worthy of Government attention, and, after careful deliberation, following the action of the State Irrigation Congress, it selected the Utah Lake project, and employed me to go to Washington and present the matter to the Department of the Interior, which I did in May last, filing with the Secretary a statement showing the claims of parties who had vested rights in the lake as a reservoir, and asking to have the Government commence work immediately for the purpose of determining the feasibility of the project from an engineering standpoint. The request met with favorable action, and the preliminary work is now in progress.

It is the purpose of the commission to do all in its power to facilitate the Government work and, when the project is found to be feasible, to assist in effecting such an arrangement as will be satisfactory to the Department of the Interior and the parties interested and will insure the early completion of the work.

The legislature also passed a new irrigation law for the State, which, it is believed, will greatly promote the interests of irrigation. It is designed to permanently settle titles to the use of water and to provide for recording and transferring the same, so that there will be a complete record of such titles; also to provide for the apportionment and distribution of water and for subsequent appropriations thereof.

The State engineer is authorized to make a complete hydrographic survey and map of each river system and water source of the State and to collect such facts as will, in his judgment, aid in ascertaining the existing rights to the use of water and in determining the volume of surplus or unappropriated water, if any, in each of such streams or sources. To him is also given the general supervision of surplus or unappropriated water.

When the State engineer has completed the hydrographic survey of any river system or water source he must file a written statement with the clerk of the district court of the county in which the same is situated; or, if the system extends into more than one county, the statement may be filed in any county that embraces any part of the system that the engineer shall select as most convenient for the water users. The statement shall set forth the completion of the survey, the names and post-office addresses of all persons using the water of the

system, so far as the same are known to the State engineer, and such other facts and information as he may deem necessary, and, on the filing of such statement, the district court in the county where the same is filed, shall have exclusive jurisdiction to determine all water rights on that river system or water source, in accordance with the provisions of the law.

Within thirty days after the filing of the statement by the State engineer the clerk of the court must give public notice that all persons claiming the right to the use of any of the water of that river system or water source must file a written statement with the clerk of the court within six months after the first publicatian of the notice, setting forth their respective claims to the use of such water. This notice is to be published in a newspaper having a general circulation, and to be mailed, by registered letter, to each of the persons whose names and addresses are given in the statement filed by the State engineer.

Each person claiming the right to use any of the water of the river system or water source, is required, within six months after the first publication of the notice, to file in the office of the clerk of the court giving the notice, a statement in writing, which must be signed and verified by the oath of the claimant, setting forth, in detail, the facts concerning his claim and appropriation, the details of which are specified in the law, and any person failing to make and deliver such statement within said time is forever barred from subsequently asserting any right theretofore acquired to the use of the water of the river system or water source in question.

At the expiration of six months after the first publication of the notice the district court in which the statements of claim have been filed may appoint a referee or referees to take testimony and determine the rights of the claimants to the use of the water. The statements stand in the place of pleadings, and the map made by the State engineer is prima facie evidence of the facts stated therein or delineated thereon. It is made the duty of the referee to take testimony, after fifteen days' notice to the claimants, at such times and places within the boundaries of the river system or water source as may be convenient to the respective claimants interested, and to determine the rights of the claimants. Contests are also provided for.

On the completion of the evidence the referee is required to state, in writing, the facts found by him as to each claim submitted, and the conclusion of law in relation thereto, and to report the same, with a form of decree, to the court, which may review the report and enter the decree thereon, or set aside, alter, or modify the same, and enter the decree as modified. The decree must determine and establish the rights of the several claimants to the use of the water, set forth the name and post-office address of the claimant, the flow (per second) of water entitled to be used, the purpose for which the water is to be

used, the time during which it is to be used each year, the name of the stream or other source from which the water is diverted, the place on the stream or other source where the water is diverted, the priority number of the right and the date of the right, and such other matters as will fully and completely define the right.

Appeals to the supreme court are provided for from decrees of the district court, and if no appeal is taken within six months after the decree has been entered it becomes final, and the clerk issues a certificate in duplicate, attested under the seal of the court, setting forth the substance of the decree. One copy of the certificate is transmitted in person or by registered mail to the appropriator and recorded in the office of the county recorder of the county in which the water is diverted from its natural channel, and the other is delivered to the State engineer and filed in his office.

The State engineer is empowered to divide the State into water divisions and subdivide the same into districts, which shall be so constituted as to secure the best protection to the water users and the most economical supervision on the part of the State. A superintendent is provided for each water division, who is to be appointed by the State engineer, with the consent of the governor. The superintendent appoints a supervisor for each water district in his division, who must be confirmed by the board of county commissioners. The superintendent has control of the district supervisors and of the apportionment of the water in the several districts in his division, under the direction of the State engineer. The supervisor apportions the water in the natural streams of his district among the several ditches taking water therefrom, according to their respective rights, under the direction of the superintendent of his division. Any person may appeal from the action of any division superintendent or district supervisor to the State engineer, who will hear and decide the case.

Any person wishing in the future to acquire the right to the use of any of the public water in the State must, before commencing the construction, enlargement, or extension of any ditch or other works for the purpose, make an application in writing to the State engineer (upon a blank to be furnished), which shall include a map, profile, and drawings. Thereupon the State engineer, at the expense of the applicant, will publish a notice for thirty days in some newspaper of general circulation within the boundaries of the river system or water source in which the appropriation is to be made, showing by whom the application is made, the quantity of water sought to be appropriated, the stream from which the appropriation is to be made and at what point on the stream, and the use for which it is to be appropriated and by what means.

Any person interested may, within thirty days after the completion of the publication of the notice, file with the State engineer, a written

protest against the granting of the application, stating the reasons therefor, which will be considered by the engineer, who will approve all applications made in proper form, where the proposed use will not impair the value of existing rights, or be otherwise detrimental to the public welfare. Where there is no unappropriated water in the proposed source of supply, or where the proposed use will conflict with existing rights, or threatens to prove detrimental to the public interest, the State engineer must reject the application. An appeal to the courts is allowed from such rejection.

The State engineer must require that actual construction work shall begin within ninety days from the date of approval, and that the construction of the proposed irrigation works shall be completed within a period of five years from the date of such approval. When it appears to his satisfaction that any appropriation has been perfected in accordance with the application therefor, he must issue a certificate, in duplicate, to the party making the same, setting forth the name and post-office address of the person by whom the water is to be used, the flow of water per second to be used, the purpose for which the water is to be used, the time during which the water is to be used each year, the name of the stream or source from which the water is diverted, the place on the stream or source where the water is to be diverted, the priority number of the right, the date of the appropriation, and such other matter as will fully and completely define the right of the person to the use of the water. One copy of the certificate is to be filed in the office of the State engineer and the other delivered to the appropriator and recorded in the office of the recorder of the county where the water is diverted from the natural stream or source.

The law declares that the water of all streams and other sources in the State, whether flowing above or under ground, in known or defined channels, is the property of the public, subject to all existing rights to the use thereof; that the discharge of 1 cubic foot per second of time, which is known as the "second-foot," shall be the unit of measurement of flowing water, and that the "acre-foot" shall be the unit of measurement of quantity; and that "beneficial use" shall be the basis, the measure, and the limit of all rights to the use of water in the State.

The law also provides that water rights shall be transferred by deed in substantially the same manner as real estate, except when represented by shares of stock in a corporation, and the deed shall be recorded in the office of the recorder of the county where the place of diversion of the water from its natural channel is situated, and that every deed of water right so recorded shall, from the time of filing the same with the recorder for record, impart notice to all persons of the contents thereof; and subsequent purchasers, mortgagees, and lien holders shall be deemed to purchase and take with notice; that every deed of water right which shall not be so recorded shall be void

as against any subsequent purchaser, in good faith and for a valuable consideration, of the same water right or any portion thereof, where his own deed shall be first recorded.

This is merely a brief outline, showing the general scope of the law, which contains many details and miscellaneous provisions that I have not enumerated.

WORK ON NORTH PLATTE RIVER IN WYOMING.

By JOHN E. FIELD.

The North Platte rises in northern Colorado and flows northward to about the center of Wyoming, where it turns and flows a little south of east into Nebraska. The waters of the main stream are used very little in Wyoming, the principal sources of supply being the tributaries, which have a much greater fall per mile, and the diversion of whose waters is much more easily accomplished. During the irrigating season, except at time of floods, almost no water reaches the main stream from them.

Considerable irrigation farming is done in the North Platte Valley in Nebraska with water from that river, so that, as the water comes from Colorado, flows through Wyoming, and is used there and in Nebraska, we may expect some interesting developments of an interstate character as to water rights along the North Platte.

As far as the projects contemplated under the reclamation law are concerned, however, there appears to be no danger of complications as regards priority of rights. One reservoir now being surveyed has a capacity sufficient to control all the flood waters in an ordinary season, giving almost absolute control of the flow of the stream and permitting the construction of ditches at any point with an assured supply. Should a canal be constructed above the reservoir the entire flow of the river could be diverted, and ditches lower down could be supplied from the stored water. For the Government canals constructed below the reservoir the stored water could be drawn off as desired.

The reason of the nonuse of the water of North Plate River is apparent after a brief examination of the district: First, the fall of the river is but 5 or 6 feet per mile; second, the land bordering the stream lies at a considerable elevation, making it necessary that a canal to cover any considerable area should be about 200 feet above the river; third, the zone between the mesa and the narrow river bottoms is very rough, and cut by gulches and arroyos, which will make any canal very crooked and construction very expensive.

The first work done consisted of running a preliminary canal line from Guernsey, in the eastern part of the State, into the Goshen Hole, to ascertain the probable cost and the area that could be irrigated.

The canal starts at a point about 1 mile above Guernsey, at the Narrows, so called on account of the river being contracted at that point by a reef of crystalline limestone, through which the water has cut an excellent location for a diversion dam. The line was started 70 feet above low water, and later increased to 85 feet, and was carried at that elevation into the Hole, the total length of the canal being 140 miles, covering about 200,000 acres, 150,000 of which are probably irrigable.

To reach a point about 17 miles below the Narrows by the river, on the south side, it was found that 33 miles of canal was required, 6 miles of which was tunnel or equally expensive cuts, and as much more of side-hill work through rock. The cost of this piece of canal appeared to be prohibitive, and a line was run on the north side. Work on this is not yet complete, but the indications are that about 4 miles of tunnel and as much more of heavy side-hill work will be eliminated, making the canal about 8 miles shorter. Under this plan, however, an inverted siphon must be built, with a capacity of at least 1,000 second-feet, and preferably of 1,500 second-feet. It must be 2½ miles in length, and under a pressure at the lowest point of 190 feet head.

The second piece of work taken up was at Devils Gate, on Sweetwater River, a short distance above its junction with North Platte River. Capt. Hiram N. Chittenden, of the Corps of Engineers, United States Army, had made a report on the site. During the fall of 1902 drilling was begun for the purpose of testing the foundations for the proposed dam. The water supply, however, was found to be very small, the maximum flow being about 400 second-feet and the minimum 5 second-feet, mostly seepage. This was very disappointing, as it showed that the construction of a large storage reservoir was not possible, on account of the small supply.

Below Devils Gate, however, is a tract of land which it was thought might be reclaimed by building a dam 50 feet high instead of 90 feet, as at first contemplated, which would impound sufficient water to irrigate probably 20,000 acres. The land is rough in places, with many alkali lakes and beds scattered over it. A topographic survey was therefore made of the area under the canal. This survey has been completed, but the classification of the land is not yet made. It is doubtful if enough good land will be covered to justify the cost of the dam, rights of way, and canals.

The third piece of work undertaken was the examination and survey of a reservoir site on North Platte River, at the mouth of the Sweetwater, about 20 miles below Devils Gate. The dam site is in a box canyon, but 80 feet wide at low-water mark, and 160 feet wide at a point 157 feet higher. The diamond drill has shown that bed rock is but 10 feet below low water and that it is solid granite, as are also the walls of the canyon. The dam as contemplated will have a total

height of 190 feet, the cubical contents being about 75,000 yards. The capacity of the reservoir is probably at least 750,000 acre-feet, possibly 1,000,000 acre-feet. The topographic survey of the reservoir has not yet been completed. These figures will show that the cost per acre-foot is remarkably small. Drilling, as stated, has been done at both the Devils Gate and North Platte sites with very satisfactory results.

The problem of obtaining a suitable body of land on which to use the water remains unsolved. At present a canal line is being run from Alcova, some eight miles below the reservoir, to ascertain the possibility of covering the mesas north and west of Casper. Should this be impracticable, surveys will be made lower down to find smaller tracts that can be irrigated by the construction of several smaller canals on each side of the river, some of which will probably extend into Nebraska.

INVESTIGATIONS IN WYOMING.

By JEREMIAH AHERN.

PROJECTS CONSIDERED.

In northern Wyoming two projects are under consideration—the Lake De Smet project, in Johnson County, and the Shoshone project, in the vicinity of Cody, in Bighorn County.

The Lake De Smet project was examined and favorably reported upon a few years ago by Capt. Hiram N. Chittenden of the Corps of Engineers, United States Army.

Soon after the enactment of the reclamation law, in June, 1902, surveys and investigations looking to the storage in general of the surplus water from the eastern slope of the Bighorn Mountains, and more particularly the storage of some of this water in the Lake De Smet basin, were begun and carried on during the season of 1902. During the winter months following preliminary plans and estimates of the probable cost of the project were made.

LAKE DE SMET PROJECT.

Lake De Smet lies in the comparatively low rolling country east of the Bighorn Mountains, being about 8 miles from their base. Its drainage basin is only a few square miles in extent, a large part of its water supply apparently being seepage water from irrigating ditches. It has no visible outlet. The lake is formed by a natural basin, the present surface of the water being about 3 miles in length, and having an average width of a little more than three-fourths of a mile, comprising an area of 1,500 acres. The lowest point of the rim of the lake basin is 30 feet above the present surface of the water, and if its

water surface should be raised to this point the lake would have a storage capacity of about 54,000 acre-feet. It is proposed to divert the surplus water of Piney Creek into this lake for storage.

Piney Creek drains a portion of the eastern slope of the Bighorn Mountains, some of its tributaries having their sources north of and some south of Cloud Peak. This peak is the highest point in the Bighorn Mountains, its elevation being somewhat over 13,000 feet. Owing to the high elevation of a large part of the drainage basin of Piney Creek, the precipitation and consequently the run-off is comparatively large, the maximum discharge of the creek being about 1,200 cubic feet per second. After leaving the mountains Piney Creek flows through a rolling country, there being no large area of bench lands that could be irrigated by its water. All of the irrigable land is along the stream and its tributaries, which are for the most part long, narrow valleys, and nearly all of it is in private ownership.

A canal necessary to divert the water of Piney Creek into Lake De Smet would be about 4 miles long, and most of it would be on ground where the cost of construction would not be high. At the present time the only uncertainty about the project is the water supply.

In August, 1902, a gaging station was established near the proposed point of diversion, so that at the end of the present season the first definite knowledge about the water supply will be recorded.

The stored water would be used for irrigating land along Piney and Clear creeks and possibly along Prairie Dog Creek in the vicinity of Sheridan. Practically all of the land along Piney Creek and about one-half of that along Clear and Prairie Dog creeks are in private ownership. All of this land lies at a comparatively low elevation for Wyoming, between 3,700 and 4,000 feet.

SHOSHONE PROJECT.

The Shoshone project contemplates the diversion of Shoshone River in the canyon about 6 miles above the town of Cody, and the storage of a portion of the surplus flow above the canyon. At the upper end of the canyon is a good dam site, the canyon at this point being about 80 feet wide with nearly vertical walls of sold granite. It has been determined that a dam 200 feet high would store 336,000 acre-feet.

The North Fork of Shoshone River rises just east of Yellowstone National Park, some of its tributaries having their sources within the park. It then flows in a general easterly direction to its junction with the South Fork, coming from the southwest and having its source southeast of the park. Shoshone River then enters a canyon about 3 miles in length, about 1,000 feet deep, and 65 feet wide in its narrowest place. A considerable portion of this canyon is through solid granite, with nearly vertical walls in many places. Near its lower end are deposits from extinct geysers and hot springs and at

a few places along the north side are large areas covered with slide rock. The discharge of Shoshone River during 1903 was more than one million acre-feet.

It is proposed to take the water out near the head of the canyon, on the north side, at such an elevation as will bring it on the high bench at the mouth of the canyon. The water will be conveyed along the side of the canyon in a suitable conduit, consisting in part of tunnels and canals. After leaving the canyon there are several wide draws or ravines that cross the irrigable land at right angles to the general direction of the canal, the crossing of which will add considerable to the cost. It is probable that the most economical way of crossing these draws will be by means of inverted siphons, constructed possibly of steel or concrete pipes.

The land that would be reclaimed by this project extends along the north or left bank of Shoshone River for a distance of about 40 miles, and comprises somewhat over 100,000 acres, nearly all of which is irrigable. This land lies at an elevation of from 4,100 to 5,100 feet. The Cody branch of the Burlington Railroad passes through it for a distance of 35 miles, the town of Cody being on the opposite side of the river, and near its upper end. The town of Garland is on the lower portion of this land. With the exception of four or five small ranches nearly all of this land has been segregated by the State under the provisions of the Cary law, but owing to the magnitude of the enterprise its promoters have been unable to obtain the necessary capital for construction. It is understood that if the Government shall undertake the construction of this project, the land will be released to the Secretary of the Interior, and that the water right will go with it.

It is not expected that there will be any complications over water rights or any difficulties to overcome in reaching an agreement with landowners. All of the public land under the project not segregated by the State has been withdrawn from entry other than homestead.

A topographic survey by plane-table methods was made of a strip along the north side of the canyon, the distance between its limiting contours being 140 feet in vertical height. The length of this strip is about 3½ miles, and its area covers all feasible locations for conduit lines. The scale of this map is 100 feet to an inch, the contour interval 5 feet. On this map the most feasible location will be made and from it all of the preliminary plans and estimates. For the primary horizontal control of this map a base line 7,170 feet in length was located on the bench below the mouth of the canyon and measured with a 300-foot steel tape. From this base a triangulation was carried into the canyon by locating stations on both sides of the canyon, so as to form well-conditioned quadrilaterals. All of the angles were then measured with a transit and the sides of the triangles were computed.

The stations were plotted on the plane table sheets by taking any one triangle and laying off one of its sides on the sheet in approximately its proper position. This, of course, gave two points of the triangle. Calling this line the base of the triangle, the third point was plotted by laying off at its proper place the altitude of the triangle. In this particular survey this method was found more convenient than computing the rectangular coordinates of each station referred to a common origin. The sheets were joined by using a line common to two sheets.

For vertical control a line of levels was carried up the canyon approximately along the top contour and then down the canyon along the lower portion of the area, thus forming a closed circuit.

SURVEY OF IRRIGABLE LAND.

The irrigable land is being mapped on a scale of 1,000 feet to an inch. The contour interval is 10 feet. The horizontal control of this survey depends on the primary triangulation carried across northern Wyoming by the topographic branch during the years 1896–1898. One of the azimuth stations of this triangulation—Corbett—is on the left bank of Shoshone River, and about 10 miles below the upper end of the irrigable land.

A point from which three of the primary stations—Hart Mountain, McCulloch, and Cedar Mountain—are visible was selected, 3¼ miles northwest of the azimuth station, and the angles at this point subtended by the three primary stations were measured with a transit reading to 30 seconds. These angles, together with the known positions of the primary stations, gave data for computing the length of the line to the azimuth station. This line was then used for a base for developing the triangulation over the area under survey. Stations were selected from 3 to 7 miles apart, and so located that well-conditioned quadrilaterals would be formed. The angles of each quadrilateral were measured with a transit reading to 30 seconds. The angles of each triangle were arbitrarily adjusted and the sides computed, also the rectangular coordinates of each station, the axes of coordinates being a meridian through the azimuth station and a line perpendicular to it. Owing to the comparatively small area, about 200 square miles, reductions taking into account the curvature of the earth were not deemed necessary. As a check on the work a side of a quadrilateral near the lower end of the work was measured roughly with a 300-foot steel tape. A connection also was made with the base line used for the canyon triangulation.

The stations were plotted on detail paper, from which they were transferred to the plane-table sheets by pricking through, the two scales being the same.

The vertical control depends on a line of primary levels of the topographic branch that crosses the area.

With the exception of the triangulation and leveling the plane table was used in all topographic work. Wherever land-office corners were found, connections with them were made.

WORK IN WASHINGTON.

By T. A. NOBLE.

The State of Washington may be divided into two sections which are wholly different in climate, soil, flora, and fauna. The western section, including the Puget Sound region, is humid, heavily timbered, very irregular in topography, and has numerous small streams flowing from the mountains into Puget Sound and the Pacific Ocean. The mean annual rainfall is somewhat over 50 inches.

The eastern section, which is the part of the State that is susceptible of improvement by irrigation, includes the great plains of the Columbia River Valley and the valleys of a number of tributaries, and is arid and semiarid. The higher portion of this section is mountainous, and bears a sparse growth of pine. It is drained entirely by Columbia River and its tributaries. The mean annual rainfall is about 16 inches.

The line between these two sections is sharply defined by the summits of the Cascade Mountains, which extend in an irregular line from the Canadian boundary southward to Columbia River, near the one hundred and twenty-first meridian.

In the central-southern portion of the eastern section are the great plains of Columbia River, known as the Big Bend country, the surface of which is an enormous lava overflow, lying between the Rocky and Cascade mountains on the east and the west, respectively, and a low range of mountains on the north, near the Canadian boundary. This plain, in its northern and eastern parts, lies at an elevation of from 2,300 to 2,800 feet above sea level and gradually slopes toward the south and east to the junction of Snake and Columbia rivers. Through this plain Columbia River has cut an irregular channel, reaching in places a depth of 2,000 feet. The Columbia, where it crosses the Canadian boundary, is about 1,300 feet above sea level. The lowest portion of this land is at the junction of Snake and Columbia rivers, where it is about 300 feet above sea level. The area lying below 1,000 feet is relatively small, so that it is physically impossible to utilize the Columbia to irrigate any large part of this land, for it would require a canal 200 miles long to take water from the Columbia River.

On the upper levels of this plain—that part lying above an elevation of 2,000 feet—is the famous wheat land of eastern Washington, comprising

system, so far as the same are known to the State engineer, and such other facts and information as he may deem necessary, and, on the filing of such statement, the district court in the county where the same is filed, shall have exclusive jurisdiction to determine all water rights on that river system or water source, in accordance with the provisions of the law.

Within thirty days after the filing of the statement by the State engineer the clerk of the court must give public notice that all persons claiming the right to the use of any of the water of that river system or water source must file a written statement with the clerk of the court within six months after the first publicatian of the notice, setting forth their respective claims to the use of such water. This notice is to be published in a newspaper having a general circulation, and to be mailed, by registered letter, to each of the persons whose names and addresses are given in the statement filed by the State engineer.

Each person claiming the right to use any of the water of the river system or water source, is required, within six months after the first publication of the notice, to file in the office of the clerk of the court giving the notice, a statement in writing, which must be signed and verified by the oath of the claimant, setting forth, in detail, the facts concerning his claim and appropriation, the details of which are specified in the law, and any person failing to make and deliver such statement within said time is forever barred from subsequently asserting any right theretofore acquired to the use of the water of the river system or water source in question.

At the expiration of six months after the first publication of the notice the district court in which the statements of claim have been filed may appoint a referee or referees to take testimony and determine the rights of the claimants to the use of the water. The statements stand in the place of pleadings, and the map made by the State engineer is prima facie evidence of the facts stated therein or delineated thereon. It is made the duty of the referee to take testimony, after fifteen days' notice to the claimants, at such times and places within the boundaries of the river system or water source as may be convenient to the respective claimants interested, and to determine the rights of the claimants. Contests are also provided for.

On the completion of the evidence the referee is required to state, in writing, the facts found by him as to each claim submitted, and the conclusion of law in relation thereto, and to report the same, with a form of decree, to the court, which may review the report and enter the decree thereon, or set aside, alter, or modify the same, and enter the decree as modified. The decree must determine and establish the rights of the several claimants to the use of the water, set forth the name and post-office address of the claimant, the flow (per second) of water entitled to be used, the purpose for which the water is to be

used, the time during which it is to be used each year, the name of the stream or other source from which the water is diverted, the place on the stream or other source where the water is diverted, the priority number of the right and the date of the right, and such other matters as will fully and completely define the right.

Appeals to the supreme court are provided for from decrees of the district court, and if no appeal is taken within six months after the decree has been entered it becomes final, and the clerk issues a certificate in duplicate, attested under the seal of the court, setting forth the substance of the decree. One copy of the certificate is transmitted in person or by registered mail to the appropriator and recorded in the office of the county recorder of the county in which the water is diverted from its natural channel, and the other is delivered to the State engineer and filed in his office.

The State engineer is empowered to divide the State into water divisions and subdivide the same into districts, which shall be so constituted as to secure the best protection to the water users and the most economical supervision on the part of the State. A superintendent is provided for each water division, who is to be appointed by the State engineer, with the consent of the governor. The superintendent appoints a supervisor for each water district in his division, who must be confirmed by the board of county commissioners. The superintendent has control of the district supervisors and of the apportionment of the water in the several districts in his division, under the direction of the State engineer. The supervisor apportions the water in the natural streams of his district among the several ditches taking water therefrom, according to their respective rights, under the direction of the superintendent of his division. Any person may appeal from the action of any division superintendent or district supervisor to the State engineer, who will hear and decide the case.

Any person wishing in the future to acquire the right to the use of any of the public water in the State must, before commencing the construction, enlargement, or extension of any ditch or other works for the purpose, make an application in writing to the State engineer (upon a blank to be furnished), which shall include a map, profile, and drawings. Thereupon the State engineer, at the expense of the applicant, will publish a notice for thirty days in some newspaper of general circulation within the boundaries of the river system or water source in which the appropriation is to be made, showing by whom the application is made, the quantity of water sought to be appropriated, the stream from which the appropriation is to be made and at what point on the stream, and the use for which it is to be appropriated and by what means.

Any person interested may, within thirty days after the completion of the publication of the notice, file with the State engineer, a written

protest against the granting of the application, stating the reasons therefor, which will be considered by the engineer, who will approve all applications made in proper form, where the proposed use will not impair the value of existing rights, or be otherwise detrimental to the public welfare. Where there is no unappropriated water in the proposed source of supply, or where the proposed use will conflict with existing rights, or threatens to prove detrimental to the public interest, the State engineer must reject the application. An appeal to the courts is allowed from such rejection.

The State engineer must require that actual construction work shall begin within ninety days from the date of approval, and that the construction of the proposed irrigation works shall be completed within a period of five years from the date of such approval. When it appears to his satisfaction that any appropriation has been perfected in accordance with the application therefor, he must issue a certificate, in duplicate, to the party making the same, setting forth the name and post-office address of the person by whom the water is to be used, the flow of water per second to be used, the purpose for which the water is to be used, the time during which the water is to be used each year, the name of the stream or source from which the water is diverted, the place on the stream or source where the water is to be diverted, the priority number of the right, the date of the appropriation, and such other matter as will fully and completely define the right of the person to the use of the water. One copy of the certificate is to be filed in the office of the State engineer and the other delivered to the appropriator and recorded in the office of the recorder of the county where the water is diverted from the natural stream or source.

The law declares that the water of all streams and other sources in the State, whether flowing above or under ground, in known or defined channels, is the property of the public, subject to all existing rights to the use thereof; that the discharge of 1 cubic foot per second of time, which is known as the "second-foot," shall be the unit of measurement of flowing water, and that the "acre-foot" shall be the unit of measurement of quantity; and that "beneficial use" shall be the basis, the measure, and the limit of all rights to the use of water in the State.

The law also provides that water rights shall be transferred by deed in substantially the same manner as real estate, except when represented by shares of stock in a corporation, and the deed shall be recorded in the office of the recorder of the county where the place of diversion of the water from its natural channel is situated, and that every deed of water right so recorded shall, from the time of filing the same with the recorder for record, impart notice to all persons of the contents thereof; and subsequent purchasers, mortgagees, and lien holders shall be deemed to purchase and take with notice; that every deed of water right which shall not be so recorded shall be void

as against any subsequent purchaser, in good faith and for a valuable consideration, of the same water right or any portion thereof, where his own deed shall be first recorded.

This is merely a brief outline, showing the general scope of the law, which contains many details and miscellaneous provisions that I have not enumerated.

WORK ON NORTH PLATTE RIVER IN WYOMING.

By JOHN E. FIELD.

The North Platte rises in northern Colorado and flows northward to about the center of Wyoming, where it turns and flows a little south of east into Nebraska. The waters of the main stream are used very little in Wyoming, the principal sources of supply being the tributaries, which have a much greater fall per mile, and the diversion of whose waters is much more easily accomplished. During the irrigating season, except at time of floods, almost no water reaches the main stream from them.

Considerable irrigation farming is done in the North Platte Valley in Nebraska with water from that river, so that, as the water comes from Colorado, flows through Wyoming, and is used there and in Nebraska, we may expect some interesting developments of an interstate character as to water rights along the North Platte.

As far as the projects contemplated under the reclamation law are concerned, however, there appears to be no danger of complications as regards priority of rights. One reservoir now being surveyed has a capacity sufficient to control all the flood waters in an ordinary season, giving almost absolute control of the flow of the stream and permitting the construction of ditches at any point with an assured supply. Should a canal be constructed above the reservoir the entire flow of the river could be diverted, and ditches lower down could be supplied from the stored water. For the Government canals constructed below the reservoir the stored water could be drawn off as desired.

The reason of the nonuse of the water of North Plate River is apparent after a brief examination of the district: First, the fall of the river is but 5 or 6 feet per mile; second, the land bordering the stream lies at a considerable elevation, making it necessary that a canal to cover any considerable area should be about 200 feet above the river; third, the zone between the mesa and the narrow river bottoms is very rough, and cut by gulches and arroyos, which will make any canal very crooked and construction very expensive.

The first work done consisted of running a preliminary canal line from Guernsey, in the eastern part of the State, into the Goshen Hole, to ascertain the probable cost and the area that could be irrigated.

the same under the laws of Congress. To so hold would uphold the view that a State might interfere with the primary disposal of the land of the National Government. When a party has obtained title to property from the National Government, the State Government has no right to destroy that title except under the power of eminent domain. The State of Montana can not step in and say, "The right to the water of Sage Creek, which the plaintiff acquired under the laws of Congress, you can not exercise in this State." This would be the taking of plaintiff's property from him without due process of law.

In the case of Willey *v.* Decker (73 Pac. Rep., 210), decided August 3, 1903, the supreme court of the State of Wyoming, considering the case of an interstate stream, held that the doctrine of priority of appropriation would apply to such streams without regard to State lines.

Referring to the decision in the case of Howell *v.* Johnson, the court said:

It is not necessary that we agree with all the reasons given by the court for the conclusion announced, nor that we assent to all the views expressed in the opinion. We think there can be little question but that it was rightly held that the plaintiff in the case had secured a right by appropriation to the waters of the stream as against a subsequent appropriator in the other State, which might be protected in the courts of such State against injury by acts occurring therein.

The court then continues:

We find nothing, therefore, in the fundamental principle of the doctrine of prior appropriation that he who is first in time is first in right, nor in the reasons that led to the establishment of the doctrine, which is opposed to the acquirement of a water right for the irrigation of lands in one State by the diversion of the water at a point in another State from a stream flowing in both States. (73 Pac. Rep., 210–221.)

It is not probable that any serious difficulties will arise in the development of the water supply and the application of the water to the reclamation of lands in cases where interstate waters are concerned, for the reason that the aim in all cases will be to make the best possible use of the waters from both an engineering and an economic standpoint, while the legal principles involved are capable of easy solution.

The operations of the reclamation service will involve a careful comparison and analysis of the irrigation laws of the various States and Territories in their application to actual conditions, and we may therefore expect a showing as to the advantages and defects of the different systems. One result to be anticipated from the experience of the reclamation service is that a method of dealing with these matters will ultimately be evolved that will afford a solution for many of the difficult problems confronting legislators in dealing with this subject and produce a code of irrigation law adaptable to the varying conditions, and that will amply protect the individual irrigator in his rights and open the way for the development of irrigation interests on a substantial basis.

The ideal of attainment in the minds of most writers and thoughtful students of the subject is the establishment of a community system of

management and operation under the control of the water user, with a guarantee of a reliable water supply, an indefeasible water right, and an interest in the entire irrigation system, with the opportunity to work out the highest agricultural development of the community. Something has been attempted in this direction by the various district irrigation laws, with comparatively small results.

The reclamation act, however, offers an opportunity for an advance toward this ideal under the auspices of the Government, with a promise of good business management and an assurance of justice and fair treatment for all. It therefore behooves all who have the best interests of irrigated America at heart, to work in harmony with the Government along the lines of operation of the reclamation act, so carefully modeled on past experience toward the attainment of this high standard of irrigation development.

ELECTRICAL TRANSMISSION OF POWER FOR PUMPING.

By H. A. STORRS.

Our interest in the electrical transmission of power arises chiefly from the following conditions: (1) In connection with the storage or diversion of the water of streams in arid regions for irrigation purposes, it is frequently found possible to develop power without greatly increasing the cost of the dam, headworks, or other parts of the canal system; (2) power so obtained may often be advantageously used for pumping water from wells, or from one canal to another at a higher level; (3) electricity is conceded to be the most suitable agent for transmitting this power from the generating station to the pumps.

As this paper must be brief, only a few of the interesting questions involved in transmitting power electrically can be mentioned.

Since a consideration of the loss of power likely to occur in a power-transmission system is of first importance I will briefly state the case for the electrical method:

Power delivered by the water wheel to the electric generator will suffer a loss during the conversion from mechanical power to electrical power of from 5 to 10 per cent; a loss during transformation from a low to a high voltage of 2 to 3 per cent; a loss during transmittal from power plant to pumping station of 5 to 10 per cent; a loss during transformation from a high to a low voltage of 2 to 5 per cent; and a loss during conversion, by the electric motor, from electrical to mechanical power of 5 to 15 per cent—a total loss chargeable to the electric system of about 18 per cent under favorable conditions and, perhaps, 35 to 40 per cent where the conditions are somewhat unfavorable. Assuming the total loss to be ordinarily 25 per cent, and assuming an average efficiency of 75 per cent for the water wheel and 70 per cent for the pump, we find that a net efficiency of about

40 per cent may be expected under ordinary operating conditions; although 50 per cent efficiency may often be obtained in an electrical-transmission system properly designed and operated at or near full load.

The elements involved in determining the proper design for the electrical system vary greatly with local conditions, but the following considerations cover some of the important points:

Regarding the choice of size and number of generating units to be installed in the power house, it may be observed that large units have the advantage of slightly higher efficiency at full load over small units, and that they cost less per kilowatt to purchase and install; but, since their efficiency falls from 3 to 5 per cent for partial loads, provision should be made by installing at least one generator of about one-half the size of the larger units, for keeping full load and on as many of the units as have to be operated.

This consideration of diminished efficiency at partial loads applies even more forcibly to most types of water wheels than to electric generators. In general, the fewest possible number of generating units should be installed, taking into account an economical design for the water wheels under the head of water available, and not forgetting that if but few units are installed the breaking down of one of them may seriously curtail the output of the system at a critical time.

Regarding the choice of line voltage for the system, it may be said that the length of line will practically determine this within certain limits; present practice would seem to favor about 50,000 or 60,000 volts for lines over 60 or 80 miles long, and approximately 1,000 volts per mile for shorter distances.

The generator voltage will usually not be over 2,500 volts, the higher voltages for use in transmission being produced in the step-up transformers; but for distances less than 15 to 20 miles a suitable line voltage of 10,000 to 15,000 volts may be generated directly in the stationery windings of the generator, thus dispensing with step-up transformers, thereby saving perhaps 20 per cent in first cost of power plant, besides raising the plant efficiency 2 to 5 per cent. At the pumping plant, also, transformers might often be omitted with these voltages, effecting a similar saving; but, probably, for motors smaller than 200 horsepower, the extra precautions required for the protection of life and property would lead to the use of step-down transformers, unless the line voltage was as low as 5,000 or 6,000 volts.

As regards the transmission line, it may be noted that the higher voltages permit the use of smaller wires, but increase the cost of pole fixtures. As is well known, the loss on a given line transmitting a given amount of power varies inversely as the square of the voltage, so that if the loss be 16 per cent for a line voltage of 10,000 volts it will be only about 4 per cent for 20,000 volts and about 1 per cent for

40,000 volts, or for the same percentage of loss the weight of the wires would be similarly diminished; but the additional precautions required with higher voltages and the increased danger of interruptions partly offset the apparent advantages.

The choice of frequency lies practically between 25 and 60 cycles per second, for the reason that all the prominent makers of power-transmission apparatus in this country have perfected designs for these frequencies. For lighting, as well as motor service, a frequency of 40 is best suited. A score of frequencies, however, ranging from 15 to 135, are now in commercial use; the lower are used solely for motor service, and those above 60 are rapidly going out of use. The lower frequencies have the disadvantage of increasing the first cost of transformers, but this is more than offset by the more favorable speeds attainable in motors and the diminished inductive effects on transmission lines. For pumping purposes a frequency of 25 is well suited, and is preferable to any higher frequency, besides affording some hope of obtaining competitive prices on standard apparatus among leading manufacturers.

Regarding the proper size, number, and arrangement of line wires, the selection must be a compromise among conflicting advantageous features. The choice of size is usually so made as to give a loss in transmission of between 5 and 10 per cent of the delivered power at full load at normal power factor. Frequently, however, it is difficult to decide whether to figure on the present full load or the future full load; that is, whether, in view of the fact that inside of three or five years the demand for power will greatly increase, thus requiring additional generators to be installed, it is best to provide at the start for this probable future increase or to design the line solely with reference to the probable conditions during the first year or two of its use, leaving future increase to be provided for by additional wires whenever required. If the latter course be chosen the question arises, Is it best to design the poles and cross arms with the expectation of placing the additional wires thereon, or to count on erecting an additional pole line when more wires are required? If provision for future extension be attempted at the start, there is the difficulty of arranging the wires symmetrically for both three and six wires on a pole. But for the increase in first cost it would be preferable to have two distinct pole lines, since by this arrangement inductive troubles between circuits would be diminished; moreover, accidents occurring on one line would probably not affect the other, and repairs could be made on one line while the other was in use, which would not usually be possible if all the wires were on one set of poles. To arrange the wires of the three-phase system symmetrically, in an electrical sense, requires that they be located at the apexes of an equilateral triangle; but in view of the numerous transpositions required on multiple-circuit lines, the

wires might about as well be placed all in the same plane, or in any other convenient position. Various ways may be adopted to secure mechanical symmetry—that is, to distribute the weights evenly on the two sides of the pole. On the whole, taking all considerations into account, a very good scheme would be to plan for the final erection of two distinct pole lines, but to erect only one pole line for use while the system was young and the other whenever the increased demand for power seemed to require it. The two pole lines would be placed 50 or more feet apart, and the three wires on each pole would be placed in the form of an equilateral triangle, with one wire on the top of the pole and the other two at opposite ends of a single cross-arm. The placing of one line wire on the pole top has the possible objection of preventing the placing of a grounded wire in this position for protection against lightning, but the need or value of such a grounded wire seems not well established, or, at least, to depend largely on local climatic conditions.

Finally, a few words about the switching devices, this general term covering all the apparatus by means of which the human agent controls the operation of the power-transmission system. Step by step, to meet the demands for higher and higher voltages, invention and development have multiplied and perfected ingenious switching devices in the endeavor to keep pace with the improvements in transformer insulation and the production of pole line insulators capable of withstanding the ever-increasing electric stresses imposed upon them, until now it appears that oil-break switches, capable of handling line voltages of 50,000 to 60,000 volts, are, or soon will be, placed upon the market and guaranteed by reliable makers. But it is not to be supposed that progress will stop here. On the contrary, another period of six years may see a doubling of line voltages similar to that which has occurred in the last six years, and with 100,000 volts power may perhaps be transmitted as cheaply and safely a distance of 200 miles as it is now transmitted a distance of 75 miles. In view of such possibilities, water powers, now apparently worthless because of their remoteness from localities where electric power might be utilized for pumping, will be brought within the scope of national works for the development of irrigation.

CORRECT DESIGN AND STABILITY OF HIGH MASONRY DAMS.

By George Y. Wisner, Consulting Engineer.

The developments of projects for the storage of flood waters for the purpose of irrigating arid lands in the Western States will require the designing of dams of unprecedented heights, and in view of the failure of similar structures which were considered safe by the engineers who

designed them, it is imperative that the plans be based upon accurate data and upon a correct determination of the nature of the stresses to which they will be subjected. If such information does not exist there should be no excuse for its not being obtained before final plans are adopted for all works where failure will result in the loss of either life or property.

With a correct knowledge of the weight, strength, modulus of elasticity, and coefficient of expansion of the materials of construction, the relation of the stresses in the horizontal arch of a dam to those in the vertical sections under varying water pressures and temperature strains, can be determined within reasonable limits, and the parts of the structure so designed that its safety will be beyond question. With the facilities that are at the disposal of the engineer for determining the strength, elasticity, and expansion of materials for the construction of large structures, it would be criminal negligence on the part of the designer not to make such determinations.

In the development of the theory of the laws that govern the action of forces in large dams most of the older investigators, as well as many engineers of the present time, have assumed that a dam is a rigid homogeneous monolith, and that the foundations, though more rigid than the structure, are still perfectly elastic. In the design of some recent structures the reverse of these assumptions has been adopted, viz, that the materials of which the structure is formed are homogeneous and perfectly elastic. Both of these assumptions certainly can not be correct, and as similar conclusions are derived in each case it hardly seems good policy to base the design of large structures on such precedents.

In regard to the elasticity of the completed dam it may be stated that the materials of which it is constructed are far from being perfectly elastic and homogeneous, and even if they were, the method of construction makes the dam a practically rigid structure. The faces of the dam are usually of cut-stone masonry with thin joints, having a modulus of elasticity of from two to five times that of the concrete or rubble filling of which the interior of the dam is composed, thus practically forming a triangular truss having a height of about one and one-third its span, which for stresses up to point of rupture through the toe of the dam may be regarded as rigid. The natural rock on which the structure should be founded is under heavy compression and is held firmly in place by the compressive forces in the earth's surface, so that elastic movement under load will be so small that the structure may be regarded as resting on a rigid base. Hence, conclusions based upon assumptions that the dam is a perfectly elastic structure, or a rigid structure resting upon an elastic foundation, will not hold true.

Under either of these assumptions the conclusion reached, which forms the basis of analysis of stresses, is that if the resultant due to the water pressure and the weight of the dam falls outside of the middle third of the base, tensile strains will be developed in the upper face of the dam, a condition which, as dams are generally constructed, is not likely to occur before the toe ruptures and the dam slides on its base.

The conclusion that tensile strains will develop in the face of a dam when the resultant passes beyond the middle third requires that the dam acts as a flexible beam bending about its central line as a neutral axis, and that with the reservoir empty there will be no pressure on the foundation at the toe and a maximum pressure at the heel, and with the reservoir full there will be no pressure at the heel and a maximum at the toe. Under the usual form of construction of dams the tendency to movement under change of water pressure is about the toe, instead of at the center of the base, and as a result the greatest stress in the structure from horizontal water pressures on the face will be parallel to and near the back face of the dam, and the pressure on the heel of the dam will not become negative when the resultant passes the middle third and probably not until the toe ruptures, which result materially modifies the actual distribution of pressure on the base from what it would be under the assumption of perfect elasticity of structure.

The fact is that there is far greater danger of failure from shearing or sliding on the base, than from tensile strains in the upper face or from overturning of the structure. The force tending to produce shearing or sliding increases with the square of the depth of the water, and in straight dams, where the tensile stresses from changes of temperature may exceed the strength of the masonry, this source of danger is a serious one, and practically limits the safe height of the structure.

While it is true that dams depending upon a gravity section for safety may require such dimensions that the line of maximum pressure in the structure will fall inside the middle third, the reasons for such sections should be based as nearly as possible upon the factors that control the action of the stresses generated in the structure by the loads to which it is subjected.

If a proper batter be given to the upstream face of a dam the maximum pressure on the foundation will be some distance from the face, and the masonry subjected to such pressures will be so situated as to withstand greater stresses than that on the surface of the structure.

In the construction of a concrete or masonry dam, the material as deposited in the structure is so pliable that for a depth of 20 to 30 feet above the foundation the pressure on all parts of the base will practically vary with the depth of material, but as the mortar sets and hardens, this foundation course acts as a beam resting on material which will yield only slightly under heavy loads, and partly distributes

throughout the foundation, the pressure arising from the weight of the upper portions of the dam.

The maximum pressure on any part of the foundation will vary with the elasticity and compressibility of the masonry and foundation, but under average conditions it should not exceed three-fourths of the weight of a column of masonry of the same height as the dam.

In short dams having ends firmly anchored in the side walls the structure acts as a horizontal beam, and the pressures developed on the base are far different from those determined by the usual method of considering the water pressure and the weight of the structure to act as a couple.

In high dams where the length near the base does not exceed 200 feet the width is so great that with the ends firmly anchored in the side walls the lower portion of the dam acts to a great degree as a horizontal beam of such magnitude that very little deformation is possible from water pressures, and consequently only a small portion of the stresses from the water pressure are transmitted to the foundation. The upper portion of the structure being longer and of less width, distortion will take place, and the stresses arising from such movement will be distributed through the base of the dam to the foundation and to the side walls, according to the design of the dam.

The strains arising from changes of temperature in straight dams are sometimes enormous, and unless provided for with large coefficients of safety may endanger the structure. The base of a dam, being massive and generally submerged in water, has much less range of temperature than the upper portions of the structure, which in many cases may exceed 100° F. The expansion of sandstone for 100° change of temperature is about 1 inch in 100 feet, of limestone about 1 inch in 125 feet, and of granite about 1 inch in 170 feet. The expansion of masonry and concrete will vary with the material of which composed, but will probably average about 1 inch in 125 feet for 100° change of temperature.

A dam 500 feet long, in passing through a change of 100° temperature, will expand or contract about 4 inches in its entire length if not prevented by the side walls, and will subject the upper portion of the wall to greater strains than those due to water pressure and weight of the dam, which the structure was designed to withstand. Internal stresses in the masonry from this cause probably have had much to do with the failure of some of the dams. A drop of 20° in temperature at the crest of the dam may cause a tensile strain of 40 tons per square foot, and unless the dam should be supported by arch construction it would be left with only its resistance to shearing and sliding to withstand the water pressure.

In constructing a dam to safely withstand the pressure of a given head of water against its face, the structure may either be made suffi-

ciently massive to resist shearing, overturning, and sliding, from the water pressure, or, if not too long a span, may be built as an arch and the stresses developed by the water pressure resisted by the side walls against which the ends of the arch abut.

It has been stated by high authorities that the masonry of a dam is so rigid and nonelastic that it can not act as an arch "for the reason that if the profile is sufficient to stand the water pressure by its own gravity the mass will not yield under pressure, and if it does not yield, no pressure can be transmitted to the sides through the curve." If this assumption is correct, is it not equally true that if the mass does not yield under the applied pressure no pressure would be transmitted in a vertical plane to the toe of the dam? The transmission of pressure to the base of the dam from horizontal water pressures on the face presupposes deformation of the structure, and if the crest be supported by an arch abutting against the side walls a portion of the pressure will be taken up by the resistance of the arch.

Since the same principle of the transmission of force is involved, whether the pressure is transmitted through the arch to the side walls or in a vertical plane to the toe of the dam, it is safe to assume that in an arch dam under normal conditions a portion of the pressure will be transmitted to the side walls and the balance to the toe of the dam, the amount of which will vary with the tensile or compressive strains in the arch arising from changes of temperature.

If the upper portion of the dam be constructed during a period of warm weather the arch will be under tension during the cooler portions of the year, and a large part of the pressure from the water will be transmitted to the toe of the dam; whereas if the walls be constructed in cool weather the arch will be under compression most of the time, and the water pressure will be transmitted directly to the side walls by the arch. It is therefore desirable that the upper portion of arch dams should be constructed during the season of the year when the temperature will average below the mean for the locality of the structure.

Assuming that the nature of the masonry in a dam will permit it to act as an arch to only a limited extent, the structure becomes practically an arch beam supported at each end, and, without regard to weight, will be safe as long as the shearing stress produced by the horizontal pressure of the water is not sufficient to produce rupture of the dam section at the side walls.

In most cases, for arch dams of less than 500 feet length on the crest, the cross section of the structure necessary to keep the pressure on the foundation within safe limits is ample to withstand the shearing stress from the water pressure, making the arch type of dam, for the same stability, much more economical than one of gravity section.

In some experiments made during the construction of the St. Louis

bridge it was found that the magnesian limestone used in the piers had practically the same modulus of elasticity as that of wrought iron, viz, 15,000,000 to 20,000,000, and although no determination has yet been made of the elasticity of the sandstone at the site of the Salt River dam, it is probable that the modulus will be at least 10,000,000.

In a dam of 25 feet or less thickness at the top, and having elasticity equal to that of ordinary rock or concrete, a large per cent of the pressure of the water against the face of the dam will be transmitted directly to the side walls by the arch, and any such dam with sufficient cross section to stand the shearing stress from water pressure on the face will be absolutely safe, without taking into consideration the weight of the dam on the foundation. In other words, if such a dam was supposed to be supported entirely by water pressure from under the foundation, the structure would still be secure.

The best construction for an arch dam is probably a wall having faces of first-class masonry with center filling of concrete and large rock. An arch dam of such construction will consist practically of two concentric arches capable of withstanding heavy pressures, connected by a water-tight concrete core, or web, having a much lower modulus of elasticity—a condition well adapted to meet all of the requirements of the problem and at the same time eliminate the necessity of molds and false works in placing the concrete in the structure. In cases of high dams of considerable length the use of railroad iron in the upper portion of the structure may be necessary to prevent temperature cracks.

The internal stresses in a large body of masonry arising from inequalities of its elasticity in its different parts, from unequal distribution of its own weight on its foundation, from compressive and tensile forces developed by changes of temperature in different parts of the structure, and from resistance to external pressures from wind, water, and shocks, can not be absolutely determined; but with a correct knowledge of the weight, strength, elasticity, and expansion of the materials with which constructed, the movement of the crest of the dam, for changes of temperature and water pressure, can be stated sufficiently near for determining thickness of the dam and the radius of the horizontal section best adapted for distributing the stresses between the base of the dam and the side walls against which the structure abuts. In any arch dam the thrust in any horizontal section is equal to the pressure on a unit of surface multiplied by the length of the radius, and the compression which takes place in the arch is equal to two times the deflection at the center into the tangent of one-fourth of the arc covered by the section considered.

The mid-channel section of the dam may be regarded as a vertical cantilever beam supported rigidly at its base and composed of material of which the weight and modulus of elasticity are known.

The application of pressure to the upstream edge of the section will cause a deformation of the structure and a slight movement of the crest downstream, which, from the known nature of materials and design of construction, can be computed within reasonable limits.

This movement of the crest corresponds with the deflection in the horizontal arch, and the section instead of being a vertical cantilever beam becomes a beam rigidly fixed at the base with its free end supported by an arch having a slight elasticity, and if the radius of the arch be properly proportioned with regard to the height of the dam, the length of the crest, and the modulus of elasticity of the materials of construction, the stresses from the water pressure will be transmitted to both the base of the dam and the side walls, practically giving the structure more than twice the stability of a gravity section dam of the same dimensions.

The height at which a dam may safely be built at any given location does not depend so much on the water pressure to which it will be subjected as upon the nature of the material of which constructed and the changes of temperature which will occur in the masonry. In a dam 250 feet high, and 500 feet long on the crest, the tensile and compressive strains due to the ordinary range of temperature may be more than four times those arising from the water pressure with the reservoir full. In a straight dam they are liable to open the joints of the masonry, and in an arch dam will cause compressive stresses in the masonry at the base and side walls as great as is safe to place on ordinary materials of construction.

In the analysis of a recent structure it is stated that precedents are of greater importance than theoretical analysis of the stresses in determining the safety of the dam. The fact that a dam has not failed is by no means a safe indication that it has been correctly designed, and to follow such methods of construction without original investigation of all the conditions involved in each case is liable to bring the engineer to grief.

While it is true that the earlier investigators in their endeavors to determine the laws governing the action of stresses in dams were obliged to make certain assumptions, the facilities afforded the engineer of the present time for determining the strength and physical characteristics of all materials of construction largely eliminates such necessity. In case any problem should arise where certain general assumptions are necessary it may be stated as a general principle that the nearer such assumptions are made to the true conditions and facts the more nearly will the results obtained conform to those which would result from an absolutely accurate determination of all the factors involved.

RECONNAISSANCE AND PLANS.

By GEORGE Y. WISNER, Consulting Engineer.

In the course of my work as consulting engineer I have made trips of inspection to the sites of proposed projects in Arizona, Colorado, California, Montana, Nevada, and New Mexico. In now reviewing the work seen certain critical suggestions as to methods and supervision of work occur to me which I wish to present for consideration, but without intent to criticise unduly.

Reconnaissance.—In some of the projects examined I noted that the preliminary field surveys have been, what seems to me, unnecessarily elaborate where the feasibility was uncertain, and in my opinion it would have been possible to determine the feasibility of the various undertakings with less outlay of time and money. Of course it must be recognized that in all Government work care must be taken, and no excuse can be offered for hasty or imperfect conclusions, and that if either extreme is to be reached, it is better to be too careful than too hasty. Again, as many of the preliminary investigations result in the condemnation of popular illusions, it is necessary to obtain absolute and incontestable facts in order to show that the statements made by others have been based upon misapprehension of the facts. Nevertheless, some care should, in my opinion, be given to planning preliminary surveys on a scale not requiring a great deal of time or money.

In making these preliminary examinations the district engineer or a competent man familiar with the requirements of the project should do the work. It should not be intrusted to younger men or persons of immature judgment, as at every step experience and quickness of perception are needed in such work. If dependence is placed upon instrumental methods, less personal judgment is required.

The general feasibility of nearly all of these projects can be determined by a careful study of the amount of precipitation on the drainage areas, and the run-off of the streams, in connection with a proper barometric reconnaissance of the streams, reservoir sites, and lands to be irrigated. By the use of three aneroid barometers on a reconnaissance, the variation in level can be determined rapidly over wide areas within limits of less than 10 feet of range in elevation, or sufficiently near to determine the general feasibility of the project from both an engineering and financial point of view. The cost of this work is small compared to that necessary for making instrumental surveys, and the investigations can be made without annoyance and antagonism as to the purposes in view.

When a project has been determined to be feasible and instrumental surveys are ordered, the investigation should be made exhaust-

ive, both as to methods and cost of putting water upon the lands to be irrigated, and as to plans and expense of properly underdraining them. The irrigation of large tracts without proper drainage from the start is almost certain to result in disappointment on the part of purchasers of homes.

Data for plans.—Some of the structures required for the projects of the service will be of unprecedented dimensions; and to be absolutely certain of their permanency, the strength and physical characteristics of the materials of construction should be carefully determined, and the nature of foundations on which they are to be based should be ascertained beyond any possible question. While such thorough investigations may not change the plans to be adopted in one case in ten, yet that tenth case might be disastrous.

Relations of consulting and district engineers.—The success of the reclamation service and the value of its work will depend largely upon harmonious relations and effort among the entire personnel, in all the divisions. To produce such a desired state of affairs will require perfect confidence on the part of the district engineers in the ability of their advisers, and upon the general confidence that the credit due to each man in developing the data on which to base plans shall be fully recognized in making the designs of the works.

The reputation of the consulting engineer is generally so well established that the credit of having made certain designs will be of little value to him, while to the young engineer in charge of the development of an important project, it may be of great importance to him whether he makes the designs and plans with the advice of the consulting engineers, or whether the data which he has collected be placed in other hands for making designs of structures, thereby indirectly implying a lack of confidence in his ability on the part of the Survey.

It is my opinion that the position of consulting engineer should be solely that of adviser, so far as the district engineers are concerned, and that in all cases the latter should be allowed the privilege and credit of developing the designs of the works under their charge, acting with the advice and guidance of their advisers.

With such a mutual understanding, the relations will always be pleasant and harmonious, while, if otherwise, more or less jealousy will be engendered. It is true that the time of the district engineer is often so taken up with other duties that the natural tendency would be to relieve him from the additional work of making plans, but when the credit of such work is necessary to building up his professional reputation, it should not be taken from him, for the reason that, outside of justice to the individual, his actual value to the service depends largely on his personal interest and enthusiasm in his work, to maintain which there must be personal credit and advancement to look forward to. The trouble that arose during the construction of the Austin dam fur-

nishes probably one of the best examples on record to show how not to conduct work, the disastrous results being a costly object lesson of the danger that may arise from lack of confidence and harmony between the consulting engineer and the engineer in charge.

Economy of water.—An excessive use of water on irrigated lands is often more injurious than an insufficient supply during occasional dry seasons; and since, where large tracts are to be colonized, the settlers will be generally ignorant of the requirements for success, it seems important that a careful study be made, under the different climatic conditions of the arid States, to determine the quantity and frequency of application of water best adapted to different crops and localities, the results of the investigations to be widely published by the Government for the benefit of settlers. In many localities the amount of land suitable for irrigation is largely in excess of that for which a sufficient volume of water is available, and therefore, from the point of view of developing the country, it becomes important to determine the minimum quantity of water that will produce the proper results at different locations and under different conditions of precipitation and climate.

REPORTS AND STATEMENTS.

By H. N. SAVAGE, Consulting Engineer.

During the last month I have had an opportunity of visiting a number of projects in the States of Idaho, Oregon, Washington, and Montana. I have been impressed with the importance of making rapid yet accurate preliminary reconnaissance examinations to determine the general feasibility and controlling factors of a project before expensive field survey parties are put to work, and particularly with the necessity of keeping the district and supervising engineers fully informed as to the exact character, progress, and results of the work being carried on by each chief of party or each individual working separately.

For the best results, monthly statements are, in my judgment, not sufficiently frequent, and I would suggest that a concise report, to consist of a few paragraphs, be compiled at the end of each calendar week and promptly sent in. Brief recommendations as to the future conduct of work should also be invited and encouraged from those reporting. Such frequent reports would aid the district engineers in formulating and perfecting the plans for the works under their charge, and at the same time would develop and indicate the engineering comprehension and efficiency of the engineers and others reporting.

I also want to call attention to the unusually complete and valuable opportunity for young engineers to fully acquaint themselves with the engineering work of the reclamation service and at the same time to acquire training and experience of the highest order. The policy already in force in the field enables every engineer to become fully

conversant with the data, its method of acquisition, and its development into plans, and if the offices of the district engineers are located near the projects, as I trust they may be, it will be feasible for all those employed to gain also a complete knowledge of construction, embracing all details of contracts, supervision, and results, as well as costs. In my judgment too much can not be said nor done looking toward mutual cooperation and mutual development in our engineer corps.

METHODS AND REPORTS.

By J. B. LIPPINCOTT.

In connection with reports and the importance of having these promptly, I wish to present the scheme that I have prepared for use in California. This is embodied in the following paragraphs. It is more or less experimental in character, but is intended to cover existing conditions.[a]

MEMORANDUM FOR REPORTS TO SUPERVISING ENGINEER.

All reports relating to work done in California (including Colorado River work) by members of the hydrographic branch of the United States Geological Survey, including the reclamation service, should be addressed to Mr. J. B. Lippincott, Supervising Engineer, 408 Byrne Building, Los Angeles, Cal. All accounts and correspondence addressed to the chief engineer should pass through the office of the supervising engineer.

The following reports should be rendered:

1. A weekly time report, showing where each employee has worked and what he has done.
2. A monthly time report, made up from the weekly time report.
3. A weekly progress report, stating results, recommendations, etc.
4. A monthly progress report.
5. A monthly report of expenditures.

1. *Weekly time report.*—Each chief of party must make a weekly time report of the services rendered by his party. Each individual working independently must also make a similar weekly report of services. These reports must be made on the filing cards furnished. They must be approved by the chief of party and mailed to the supervising engineer promptly. This report should show concisely the manner in which the person was occupied and will constitute his personal record of work in the offices of the supervising engineer and of the chief engineer at Washington. The card should show by dates the kind of occupation for each day of the week. The instructions on the back of the monthly report (Form 9–245) should be followed in filling out these cards. If a field party or an individual is employed on more than one project during the week, a separate card must be made out for each project.

2. *Monthly time report.*—Before sending the weekly time report to the supervising engineer each individual working independently should fill out from it that portion of the monthly report (Form 9–245) covering the week, so that the monthly report will be an exact copy of the several time reports covering the month. At the end of the calendar month the monthly report should be forwarded to the supervising engineer, accompanied by a monthly progress report and a diagram showing where the work was done, as will be described below.

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[a] See also pp. 45–50.

A chief of party need not fill out a monthly report (Form 9–245) for each man in his party, but should send in one for himself and the party. It should be headed, "Report of services rendered by ..... and party during the month of ....., 1904." (Use Form 9–245.)

3. *Weekly report to supervising engineer.*—The engineer in the field should keep himself thoroughly informed as to the details and progress of the work in hand. To maintain the greatest possible efficiency the supervising engineer should be fully informed as to the progress, results, needs, and cost of each project, and in order that he may obtain this information it is desired that a short report be rendered once a week. It will be well to forward this report with the weekly time-report cards.

This report should be made by each chief of party, or by each individual working independently in the field, and should state concisely, in a few paragraphs, what has been accomplished since the last report was made. If work has not progressed satisfactorily for any cause, the reason for and extent of the delay should be stated.

Recommendations concerning the work in hand, the completeness of the surveys, additional investigations required or suggested, or other matters affecting the value of the work should be made when necessary. Estimates should be made of the time necessary to complete the work under way, when it is nearing completion, in order that arrangements may be made for transferring men and material to other places. Recommendations and suggestions are particularly desired.

A general rule regarding these weekly reports is that they should relate principally to the progress of the work which comes directly under the care of the supervising engineer. Anything concerning purchases, vouchers, instruments, the detailing of work to assistants, and the general administrative work should not be embodied in these reports.

Nothing in these instructions is to be construed as interfering with correspondence with the supervising engineer at any time during the week. Much of the detail office work regarding supplies, instruments, etc., is attended to by his assistants. The correspondence of the office is so large that it is desired to separate the reports from the mere detail work of the office. Concise accounts of work done will greatly assist in this matter.

3. *Monthly progress report.*—A monthly progress report, accompanied by the monthly time report and a sketch map (base for which will be furnished), should be rendered to the supervising engineer at the end of each month by each chief of party and each individual working independently in the field. After they have been inspected by the supervising engineer these reports will be forwarded to Washington for the information of the chief engineer.

The monthly progress report must be concise, comprising but a few paragraphs, and must refer strictly to the progress of the work and the results accomplished since the last monthly progress report was made, all statements of details being reserved for the weekly reports, as noted above. If work has been prosecuted on more than one project, a separate report will be required on each, and the amount of work done on each must be clearly set forth.

In case of surveys for reservoir sites the report should state approximately the number of square miles surveyed, scale of map, contour interval, character of country (as rough, broken, wooded), etc. In reporting on leveling, the number of miles run and number of bench marks established, etc., should be stated. In reporting on canal surveys, the number of miles run, whether location or preliminary, the number of miles contoured, the character of country, whether rough, wooded, etc., should be stated. In reporting on drilling for bed rock at a dam site, the number of holes sunk, the aggregate number of feet driven, and the number of feet of diamond drilling should be stated, and a general statement should be added

as to the suitability of the location as a dam site so far as developed. In reporting stream-gaging trips the routes traversed should be stated, and the number of gagings made and where they were made. In reconnaissance work a general account should be given of the route traversed, the localities visited, and the conclusions drawn. Recommendations for the abandonment of old gaging stations or establishing of new ones are desired.

Each chief of party or man working independently will be supplied with a sketch map of the locality in which he is working, on which he should indicate the areas surveyed, the routes traveled, etc. The monthly progress report, map, and monthly time report (Form 9-245) should be pinned or bound together and forwarded promptly to the supervising engineer. Exception to this rule will be made only when men are engaged in extensive reconnaissance or in stream-gaging trips.

4. *Monthly report of expenditures and cost of work.*—At the end of the month each chief of party or individual working independently in the field, in charge of work, such as topography, leveling, canal surveys, borings at dam sites, etc., shall submit to the supervising engineer an estimate of the amount of work done and the total expenses incurred for such work during the month. He shall also estimate as closely as possible the cost per unit for the work, as per square mile for topography, per foot for borings, per mile for canal surveys, etc. At the completion of the survey detailed estimates of the total cost and unit cost of work will be required from each chief of party.

In case several projects are under way at the same time and are paid for out of a general fund, the chief of party will apportion the cost among the different projects as closely as possible. It is realized that this is not precisely accurate, but it will closely approximate the actual cost.

IRRIGATION SURVEYS AND THE USE OF THE PLANE TABLE.

By J. B. LIPPINCOTT.

It is the purpose in this discussion to give a brief account of some work done with the plane table several years ago for the Bear Valley Irrigation Company in San Bernardino County, Cal. It is not claimed that the methods described are new, except in a few instances, but because so much topographic work is done by methods that seem to those who understand the plane table as being painfully crude, this discussion may be interesting and perhaps useful.

The Bear Valley Irrigation Company surveyed, and in part constructed, a canal from Santa Ana Canyon to the town of Alessandro, a distance of about 40 miles. The canal was to be a cemented channel, designed to carry about 240 cubic feet per second. The plan adopted for these surveys was to first run a preliminary transit angle line for the control of a topographic map on which the location was to be plotted. Three methods of topographic mapping were tried:

1. Contours were located in a few instances with slope board and rod, the party consisting by this method of three men. The most ground was covered in this way, but the least accuracy was obtained.

2. Rodmen were located on given contours at right angles to the stations by a level man, using a hand level and obtaining his eleva-

tions from the stations. Measurements were then made to the rods from center, and all notes were recorded by the topographer in his notebook. The party in this case consisted of a topographer, level man, two rodmen, and one or two tape men. The notes were sent to the office, where a draftsman compiled the map.

3. The plane table was used. The preliminary line had been run and the line plotted and all elevations recorded on the plane-table sheet. The board was set up over a convenient stake or hub, the level man carried his elevation from the bench marks, giving a check on these as he went, the rods were placed on contours and sighted with the alidade, sights being taken with care and not allowed to be over 300 feet long. Philadelphia rods were used.

In a country where the slope is somewhat even and about 2 horizontal to 1 vertical with canyons from 30 to 70 feet deep every 300 to 500 feet and with no brush, the speed with a party of five (topographer, level man, two rodmen, and a general helper for umbrella man) will be from 2,500 to 4,000 feet a day, actually locating four 10-foot contours and sketching five or six more, and interpolating the 5-foot contours. In the same country the speed with method No. 2, locating the same number of contours, will be about the same, reading being taken at each station. With the slope board the speed will be doubled, with readings at every station. There will, however, be a marked difference in the quality of work.

Suppose we have the preliminary line running up to and crossing canyons 50 or 60 feet deep and say 200 feet wide. There may be the opportunity of selecting a crossing from two or three different points or of running approximately around on the contour. The plane table has a 600-foot range before leaving the center line. By a traverse the plane table may occupy a series of new points at any distance from the line, giving new points of observation, from which may be made a detail survey of the canyon quickly and closely. By the use of the vertical angle the scope of the work is greatly increased. As for the slope board, in a case of this sort it is helpless. Its range is only 15 or 20 feet and the topographer can make only a rough sketch by the eye of remaining portions.

With the second method the conditions are improved by the use of a hand compass, and the tape traverse line may be carried up a canyon, and the elevations taken along with the hand level, but the work will be of the crudest nature. There are no stadia wires to take readings across the canyons and no vertical arc to assist in extending the information desired, and most of all, the great advantage of skilled and intelligent sketching directly on the sheet at the time of observation, assisted by numerous located points is lost.

It has been the verdict of topographers on the United States Geological Survey that the only way to get an expressive and accurate

topographic map is to plot and sketch it in the field. Where field notes are turned over to an office man, or even where the notes are recorded and subsequently developed by the same man, it is believed that numerous errors are unavoidable, and that nearly all the expression is lost.

The same argument pertains with especial force to places where a tunnel is proposed. The possibilities of several positions can be studied on the plane-table sheets, including in many cases, the length of line around the point, a condition that could, with difficulty, if at all, be covered by either of the other two methods of procedure. Some may argue that all these conditions ought to be shown by additional preliminary lines. This may be true and frequently the work is done in this way, but it takes nearly as long and a larger party to run the preliminary line as to run four or five 10-foot contours, and the information from the bare line is not nearly so complete. I might state that it has been my experience that a line of this nature may be run by plane-table traverse around a canyon or point a distance of from 2,000 to 4,000 feet and may close on the board within from 1 to 3 feet, the work being on a scale of 50 feet to 1 inch.

When the country is open and smooth, without canyons, from 5,000 to 6,000 feet a day can be made, locating, say, about 6 contours of 5-foot interval and sketching the 2-foot intervals.

In running through a very broken mesa country, and usually across drainage lines, the slopes being about 1 to 1, with everything covered by a dense growth of brush, an average speed of about 1,700 feet a day was maintained, and at this time, as at others, a careful map was made, frequently from 500 to 1,000 feet on each side of the line, in order to give information concerning several routes.

Unfortunately, location lines were plotted only on the topographic sketch that was drawn on the plane table, so that an absolute comparison of accuracy between the different methods can not be given at present. However, the plane-table work has stood severe tests. The average error of the contour map is about 6 inches. These errors came in principally on crossing canyons, and are on the steep slopes; on the benches and uniform slopes the average error is about 3 inches.

With a map of this nature a very detailed study can be made of various grades, and from the slopes shown an accurate estimate of the cost of the work can be had. The cost of this plane-table work is about $18.50 for each working day. In country such as was first described, with a speed of 3,500 feet a day, this will amount to say $28 a mile; in smooth, open country, at 5,000 to 6,000 feet a day, we have a cost of say $18 a mile; and in the country last described, where all the conditions are about as bad as possible, the cost will be about $57.50 a mile.

The cost of taking topography by the second method, i. e., with hand level, when we consider the increased expense of office work, is just about the same, and the results are much less satisfactory than with the plane table. A portion of the survey for the Santa Ana canal is along the face of a box canyon, which is practically a vertical rock wall. In cases of this kind topographic work had to be done largely by the use of vertical angles, the rods being taken wherever they could be held and as many points as possible located by intersection, the topography being in the nature of a general sketch. In this locality the preliminary line was run as near to an assumed grade as possible, but triangulation was frequently resorted to.

On the general topographic map a grade line was plotted and men were put to work to clear out a trail as near this line as possible. This line was then run out and plotted on plane-table sheets of uniform size on a scale of 30 feet to 1 inch. The sheets were put on a board without a tripod and taken on the line. The rods were placed on contour with hand level and measurements taken to them, the points being plotted on the board and sketching done at the same time. Measurements of this nature were taken at short intervals, and the topography was put in with precision. The final location was then plotted on these sheets. This method gives very good satisfaction in country of this nature.

In undulating rough or side hill country a plane table survey in detail as described above is believed to be justifiable, but for a flat or uniform plain probably satisfactory results can be obtained at less cost by eliminating the contour map and trusting to angle lines alone.

THE USE OF ALKALINE WATERS FOR IRRIGATION.[a]

By THOMAS H. MEANS,

In charge of alkali reclamation work for the Bureau of Soils of the Department of Agriculture.

During the summer of 1902 a representative of the Bureau of Soils visited the oases of the Oued Rihr country in the Desert of Sahara, in eastern Algeria. In these oases artesian waters carrying very large quantities of soluble matter are used successfully for irrigation. From the information gathered there, and from experience in this country, it seems that the amount of soluble matter allowable in an irrigation water has been greatly underestimated by American writers, and that many sources of water which have been condemned can be used with safety and success, provided the proper precautions are taken to prevent the accumulation of the salts. As the precautions are those which should be taken by every irrigator, even if pure water is used,

[a] Extract from Circular No. 10 of the Bureau of Soils, July 1, 1903.

it seems an important matter to bring before the American people the methods in use in the Sahara.

The staple crop grown by the Arabs in the oasis country is the date, the fruit of a palm tree known to be one of the plants most resistant to alkaline or saline conditions of the soil, but in addition to this considerable quantities of deciduous fruits, garden vegetables, and alfalfa are produced for home consumption.

Some of the vegetables successfully grown are those considered sensitive to alkali, and yet they are irrigated with water containing in some instances as much as 800 parts of soluble salts to 100,000 parts of water, sometimes as high as 50 per cent of the salts being sodium chloride.

The limit of concentration for irrigation water in the United States, even where only the most resistant field crops are to be grown, has been placed by some authorities at 30 parts sodium chloride (common salt) or sodium carbonate (black alkali), and at from 170 to 300 parts of the less harmful salts, per 100,000 of water. The Bureau of Soils, however, several years ago insisted that water of a somewhat higher salt content might be used if the soil had good natural drainage, or artificial drainage were supplied, and the methods of irrigation were modified to suit the different conditions. Thus in 1899[a] the following statements were made after a study of soils and alkali conditions in the Pecos Valley, New Mexico:

> Next to the ownership of the land and the labor questions the most important cause of the trouble in the larger portion of the Pecos Valley is the high salt content of the irrigation water, especially in certain seasons.
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> At Roswell the principal water supply contains about 76 parts of soluble matter in 100,000 parts of water. At Hagerman this is increased to about 200 parts, at Carlsbad to 240 parts, at Florence to 280 parts, at Red Bluff to 316 parts, at Pecos City, Tex., to 400 parts, and below Pecos City to over 500 parts. Five hundred parts of soluble matter may be taken as the extreme limit of endurance for plants, while 250 or 300 parts mark the danger point at which the results of the use of the water are very uncertain.
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> The limit of endurance for most cultivated plants in a water solution is about 1 per cent, or 1,000 parts of the readily soluble salts in 100,000 parts of water, but it must be remembered that in field culture the water is applied to soils already containing more or less of these salts, and also that evaporation and consequent concentration immediately set in after the application of water. It was found at Carlsbad that about 300 parts of soluble matter per 100,000 parts of water marked the extreme limit of safety of the use of water at that place.
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> One thing should be said in connection with the Carlsbad area, which seems rather anomalous in view of the statements of other investigators, namely, that with a water supply so near the limit of crop endurance as this becomes at times and in those areas in which there is already a large accumulation of salts, economy in the use of irrigation water, which is generally recommended in alkali regions, is one of

[a] Rept. 64, Field Operations, Division of Soils, 1899.

the worst methods which can be practiced. Where the soil contains a relatively large amount of salt and but little water containing much salt is frequently applied, the ordinary evaporation will increase the salt content of the soil to such an extent that crops can no longer survive; whereas if adequate drainage is provided, and a large amount of water is used, the excess of salt resulting from the evaporation of previous applications of water may be removed, and the soil moisture be maintained at nearly the same concentration as the water supply.

The fact that the Arabs in Algerian oases are actually growing sensitive plants by the aid of irrigation waters containing from 400 to 800 parts of soluble salts, in some instances 50 per cent sodium chloride, shows that the Bureau has been on the conservative side in its estimates, and should encourage a more hopeful feeling among people occupying areas where only alkali water is available for irrigation.

The prerequisite to the use of water of high salt content in irrigation is the knowledge that the methods employed are opposed to the teachings of most American writers on the subject. Those who place low the limit of safety for alkaline irrigation waters have taught that where water was badly alkaline irrigation should be sparing. They have not insisted on thorough drainage, and they have warned irrigators against too frequent irrigation. With such practices the limit of concentration which they set is probably high enough, and even then all except the most sandy soils or those with exceptionally good natural drainage would ultimately be damaged.

The methods in the oases are quite different. The Arab gardens are divided into small plots about 20 feet square, between which run drainage ditches dug to a depth of about 3 feet. The soils being very light and sandy, this ditching at short intervals insures the most rapid and thorough drainage. Irrigation is by the check method, and application is made at least once a week, though often two wettings a week are deemed necessary. A large quantity of water is used at each irrigation. Thus a continuous movement of the water downward is maintained, there is little opportunity for the soil water to become more concentrated than the water as applied, and the interval between irrigations being so short but little accumulation of salt from evaporation at the surface takes place. What concentration or accumulation does occur is quickly corrected by the succeeding irrigation.

The native gardens are situated in the date palm groves and the vegetables and fruit are grown in the partial shade cast by these trees. The natives not only have the question of very saline irrigation waters to contend with, but the soils originally are often very alkaline. In three years they reclaim land too salty to grow the minor crops, using the saline water for that purpose, following the same plan of drainage and weekly irrigation as where crops are growing. One garden situated on the side of a salt flat and originally very saline was visited. Here alfalfa was in very good condition, and fig, pomegranate, melon, tomato,

cabbage, pepper, and other plants were growing luxuriantly. The reclamation of this plat by irrigation twice a week had taken three years.

The native gardeners exercise great judgment in adapting their crops to the soil conditions. Where most alkaline the date palm alone is found; in other parts of the grove are grown the fig and pomegranate, and in other parts the vegetables less resistant to alkali. The amount of salt in the soil sufficient to injure the palm was not determined, but the French were unsuccessful in an attempt to establish a grove, the water used in irrigation being taken from a salt pond and containing, according to field determinations, 3 parts sodium carbonate, 5 parts sodium bicarbonate, and 1,036 parts sodium chloride in 100,000 parts.

The irrigation water is all drawn from artesian wells. Field tests showed as high as 816 parts to 100,000 water in actual use on soils growing vegetables.

Enough has been written to show that much more can be done with alkaline irrigation water than has hitherto been generally thought possible, and while the intensive culture of the Algerian gardeners is not suited to our western conditions, there is believed to be no reason why their methods can not be adapted, even with an improvement in point of economy, to more extensive cropping of the soil.

The soils in the Pecos Valley are not unlike those of the oases, and the general conditions in the two places are somewhat similar, although the irrigation water, even at the lowest stage of the Pecos River, is not as alkaline as the artesian water of the desert, nor are the soils naturally as alkaline. Employing the Algerian method of frequent irrigation with generous amounts of water on land well drained by open ditches or tile-drains, large areas now out of cultivation might be used to grow alfalfa, truck, and other crops.

It is believed that a comprehension of these facts will be of great value to the people of this country residing in regions affected with alkali, and that it will result ultimately in bringing into cultivation much land that has hitherto been thought worthless because of the character of the water available for irrigation.

ACCOMPANYING PAPERS.

THE RECLAMATION LAW.

The full text of the reclamation law dated June 17, 1902,[a] is as follows:

AN ACT Appropriating the receipts from the sale and disposal of public lands in certain States and Territories to the construction of irrigation works for the relamation of arid lands.

Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled, That all moneys received from the sale and disposal of public lands in Arizona, California, Colorado, Idaho, Kansas, Montana, Nebraska, Nevada, New Mexico, North Dakota, Oklahoma, Oregon, South Dakota, Utah, Washington, and Wyoming, beginning with the fiscal year ending June thirtieth, nineteen hundred and one, including the surplus of fees and commissions in excess of allowances to registers and receivers, and excepting the five per centum of the proceeds of the sales of public lands in the above States set aside by law for educational and other purposes, shall be, and the same are hereby, reserved, set aside, and appropriated as a special fund in the Treasury to be known as the "reclamation fund," to be used in the examination and survey for and the construction and maintenance of irrigation works for the storage, diversion, and development of waters for the reclamation of arid and semiarid lands in the said States and Territories, and for the payment of all other expenditures provided for in this Act: *Provided*, That in case the receipts from the sale and disposal of public lands other than those realized from the sale and disposal of lands referred to in this section are insufficient to meet the requirements for the support of agricultural colleges in the several States and Territories, under the Act of August thirtieth, eighteen hundred and ninety, entitled "An Act to apply a portion of the proceeds of the public lands to the more complete endowment and support of the colleges for the benefit of agriculture and the mechanic arts, established under the provisions of an Act of Congress approved July second, eighteen hundred and sixty-two," the deficiency, if any, in the sum necessary for the support of the said colleges shall be provided for from any moneys in the Treasury not otherwise appropriated.

SEC. 2. That the Secretary of the Interior is hereby authorized and directed to make examinations and surveys for, and to locate and construct, as herein provided, irrigation works for the storage, diversion, and development of waters, including artesian wells, and to report to Congress at the beginning of each regular session as to the results of such examinations and surveys, giving estimates of cost of all contemplated works, the quantity and location of the lands which can be irrigated therefrom, and all facts relative to the practicability of each irrigation project; also the cost of works in process of construction as well as of those which have been completed.

SEC. 3. That the Secretary of the Interior shall, before giving the public notice provided for in section four of this Act, withdraw from public entry the lands required for any irrigation works contemplated under the provisions of this Act, and shall

[a] Stat. L., vol. 32, p. 388.

restore to public entry any of the lands so withdrawn when, in his judgment, such lands are not required for the purposes of this Act; and the Secretary of the Interior is hereby authorized, at or immediately prior to the time of beginning the surveys for any contemplated irrigation works, to withdraw from entry, except under the homestead laws, any public lands believed to be susceptible of irrigation from said works: *Provided*, That all lands entered and entries made under the homestead laws within areas so withdrawn during such withdrawal shall be subject to all the provisions, limitations, charges, terms, and conditions of this Act; that said surveys shall be prosecuted diligently to completion, and upon the completion thereof, and of the necessary maps, plans, and estimates of cost, the Secretary of the Interior shall determine whether or not said project is practicable and advisable, and if determined to be impracticable or unadvisable he shall thereupon restore said lands to entry; that public lands which it is proposed to irrigate by means of any contemplated works shall be subject to entry only under the provisions of the homestead laws in tracts of not less than forty nor more than one hundred and sixty acres, and shall be subject to the limitations, charges, terms, and conditions herein provided: *Provided*, That the commutation provisions of the homestead laws shall not apply to entries made under this Act.

SEC. 4. That upon the determination by the Secretary of the Interior that any irrigation project is practicable, he may cause to be let contracts for the construction of the same, in such portions or sections as it may be practicable to construct and complete as parts of the whole project, providing the necessary funds for such portions or sections are available in the reclamation fund, and thereupon he shall give public notice of the lands irrigable under such project, and limit of area per entry, which limit shall represent the acreage which, in the opinion of the Secretary, may be reasonably required for the support of a family upon the lands in question; also of the charges which shall be made per acre upon the said entries, and upon lands in private ownership which may be irrigated by the waters of the said irrigation project, and the number of annual installments, not exceeding ten, in which such charges shall be paid and the time when such payments shall commence. The said charges shall be determined with a view of returning to the reclamation fund the estimated cost of construction of the project, and shall be apportioned equitably: *Provided*, That in all construction work eight hours shall constitute a day's work, and no Mongolian labor shall be employed thereon.

SEC. 5. That the entryman upon lands to be irrigated by such works shall, in addition to compliance with the homestead laws, reclaim at least one-half of the total irrigable area of his entry for agricultural purposes, and before receiving patent for the lands covered by his entry shall pay to the Government the charges apportioned against such tract, as provided in section four. No right to the use of water for land in private ownership shall be sold for a tract exceeding one hundred and sixty acres to any one landowner, and no such sale shall be made to any landowner unless he be an actual bona fide resident on such land, or occupant thereof residing in the neighborhood of said land, and no such right shall permanently attach until all payments therefor are made. The annual installments shall be paid to the receiver of the local land office of the district in which the land is situated, and a failure to make any two payments when due shall render the entry subject to cancellation, with the forfeiture of all rights under this Act, as well as of any moneys already paid thereon. All moneys received from the above sources shall be paid into the reclamation fund. Registers and receivers shall be allowed the usual commissions on all moneys paid for lands entered under this Act.

SEC. 6. That the Secretary of the Interior is hereby authorized and directed to use the reclamation fund for the operation and maintenance of all reservoirs and irrigation works constructed under the provisions of this Act: *Provided*, That when the payments required by this Act are made for the major portion of the lands irrigated

from the waters of any of the works herein provided for, then the management and operation of such irrigation works shall pass to the owners of the lands irrigated thereby, to be maintained at their expense under such form of organization and under such rules and regulations as may be acceptable to the Secretary of the Interior: *Provided*, That the title to and the management and operation of the reservoirs and the works necessary for their protection and operation shall remain in the Government until otherwise provided by Congress.

SEC. 7. That where in carrying out the provisions of this Act it becomes necessary to acquire any rights or property, the Secretary of the Interior is hereby authorized to acquire the same for the United States by purchase or by condemnation under judicial process, and to pay from the reclamation fund the sums which may be needed for that purpose, and it shall be the duty of the Attorney-General of the United States upon every application of the Secretary of the Interior, under this act, to cause proceedings to be commenced for condemnation within thirty days from the receipt of the application at the Department of Justice.

SEC. 8. That nothing in this act shall be construed as affecting or intended to affect or to in any way interfere with the laws of any State or Territory relating to the control, appropriation, use, or distribution of water used in irrigation, or any vested right acquired thereunder, and the Secretary of the Interior, in carrying out the provisions of this act, shall proceed in conformity with such laws, and nothing herein shall in any way affect any right of any State or of the Federal Government or of any landowner, appropriator, or user of water in, to, or from any interstate stream or the waters thereof: *Provided*, That the right to the use of water acquired under the provisions of this act shall be appurtenant to the land irrigated, and beneficial use shall be the basis, the measure, and the limit of the right.

SEC. 9. That it is hereby declared to be the duty of the Secretary of the Interior in carrying out the provisions of this act, so far as the same may be practicable and subject to the existence of feasible irrigation projects, to expend the major portion of the funds arising from the sale of public lands within each State and Territory hereinbefore named for the benefit of arid and semiarid lands within the limits of such State or Territory: *Provided*, That the Secretary may temporarily use such portion of said funds for the benefit of arid or semiarid lands in any particular State or Territory hereinbefore named as he may deem advisable, but when so used the excess shall be restored to the fund as soon as practicable, to the end that ultimately, and in any event, within each ten-year period after the passage of this act, the expenditures for the benefit of the said States and Territories shall be equalized according to the proportions and subject to the conditions as to practicability and feasibility aforesaid.

SEC. 10. That the Secretary of the Interior is hereby authorized to perform any and all acts and to make such rules and regulations as may be necessary and proper for the purpose of carrying the provisions of this act into full force and effect.

SCOPE OF RECLAMATION LAW.

The first step in any action taken under the law is a request from the engineers of the reclamation service, made through the head of the organization, the Director of the Geological Survey, to the Secretary of the Interior, for the withdrawal from entry of certain specified lands with a view to their examination in the field, in order to determine the practicability of constructing irrigation works for reclaiming them. When this request is approved by the Secretary of the Interior, the proper steps are taken, by the General Land Office, for

the withdrawal of the lands through the local land office for the district in which they are located. Thereafter no entries will be allowed, except under the provisions of the homestead law, as modified by the limitations and conditions of the act. In accordance therewith they may be limited to an area as small as 40 acres, and will not be subject to the commutation provisions of the homestead law. The entryman may, therefore, be required to reduce the area of his entry to such limit as in the opinion of the Secretary of the Interior may be reasonably required for the support of a family, and will be called upon to pay the charges per acre which may be determined on in not more than ten annual installments.

As soon as possible after such withdrawal, engineers are instructed to make the survey of the lands and of proposed canals and reservoirs; also to conduct necessary engineering investigations concerning the water supply and the conditions under which the construction will be carried on. Upon the completion of this work the results are to be summarized and submitted to the Secretary of the Interior, with such recommendations as may be deemed advisable by the Director of the Geological Survey, together with maps, plans, and estimates of cost.

It will then be the duty of the Secretary of the Interior to determine whether or not such project is practicable and advisable. If determined to be impracticable or inadvisable, he will restore to the public domain the lands withdrawn, and they will become subject to the public-land laws, as if such withdrawal had never been made. If the project is approved by the Secretary of the Interior, he may cause contracts to be let for the construction of the proposed works, either as a whole or for such portion or section as will constitute a complete system, if it should be deemed best not to undertake the entire project at that time. He will thereupon give public notice of the land which may be reclaimed thereunder, and of the particular limitations contemplated by the law as to area, cost, number of installments, date of payments, etc.

As soon as it shall be possible to furnish water for the irrigation of any particular portion of the lands involved, the entrymen thereon will be allowed to take the water; and as construction progresses additional lands will be supplied from time to time.

When payments have been made in full for the major portion of the lands irrigable under any system, the management and operation of the irrigation works will pass to the owners of the land irrigated therefrom, to be maintained at their expense, under such form of organization and under such rules and regulations as may be acceptable to the Secretary of the Interior; it is provided, however, that the title to the reservoirs and the works necessary for their protection and operation shall remain in the Government, and that they shall continue under the control and management of the Government unless otherwise provided by Congress.

MEASUREMENT OF FLOW OF STREAMS.

By E. C. MURPHY.

The minimum yearly flow of streams for periods of about a week is very important, and resident hydrographers should make special effort to obtain it on all important streams. It is not the smallest quantity that flows past the station per year that is desired, but the average natural flow for about a week when the flow is least.

On nearly all important streams a part of the natural flow is held back in ponds for a few hours a day and on the smaller streams, in some cases, for two or more days at a time and then allowed to flow through water wheels in a comparatively short time. Ordinarily, the pond is drawn down below the crest of the dam during the day and fills up again during part of the night. The proper time to make a measurement of the flow in a case of this kind is early in the morning, before the water wheels are put into use and while the water is still flowing over the crest of the dam.

The gage reader should read the gage carefully to hundredths of a foot twice a day, once when the wheels are running and once when they are not running. He should also keep a record of the number of hours a day that the wheels are in use.

The low-water conditions at each gaging station on which the accuracy of the work depends should be carefully described, such as number of channels, depth, width, velocity, and roughness of each channel, as well as the effect of dams, canals, etc., on the flow.

No gaging station should be established without the written consent of the chief engineer. A request for a station should be addressed to the director of the Geological Survey and sent to the resident hydrographer, who will examine the need therefor, the proposed site, and report to the chief engineer with recommendations. The selection of gaging stations is the most important part of the resident hydrographer's work, and should not be entrusted to an assistant.

The first and most important question to settle at a proposed station is, will the results obtained be of a reasonable degree of accuracy for all river stages. If not, the station must not be established at this place.

Ordinary flow and minimum flow results are, as a rule, of more economic value than those for high water; hence the low-water conditions at a proposed station determine whether it is a good one or not. It follows also that the proper time to seek new gaging stations on a stream is during low water. A station may appear to be a good one at high water but prove to be a very poor one at low water, on account of sluggish flow.

No discharge measurements should be made where the mean velocity is less than half a foot per second in 85 per cent of the cross section.

When it is found that the mean velocity is less than half a foot per second in 85 per cent of the cross section, the station must be abandoned unless a section having a velocity of one foot per second or more can be found within a reasonable distance from the station that can be used as a measuring station during low water.

A station should not be in the influence of backwater from another stream or lake, and should not be in the influence of a pond that may be drawn down below the level of its spillway.

A station should not be located at a bend in a stream nor at a bridge that crosses the stream obliquely, unless the piers of the bridge are parallel with the trend of the stream.

The stream bed at a proposed station should be fairly regular in shape and the velocity well distributed through the cross section. No station should be established unless at least one discharge measurement has been made and it is seen that the shape of the cross section and the distribution of velocity are satisfactory.

Resident hydrographers should remember that records of flow for short periods, except minimum flow records, are of comparatively little value. The records should cover a period sufficiently long to include the extremes of run-off. The stations should therefore be selected and equipped so that little or no change in them will be necessary for a period of several years.

The resident hydrographer must keep in mind also the cost of maintaining the station and the economic value of the results obtained, as well as the accuracy of the results. The cost of equipping a station is, on an average, about $25, and the cost of maintenance for one year is about $110, making a total for the first year of about $135. The cost of maintenance being large, the number of important streams of large size being great, and the funds at disposal being limited, the Survey can not maintain stations on small streams, the data from which are only of small and local value. In the case of a small stream the data desired is the flow for a week or month per year when the flow is least. This data can be obtained at small expense without establishing a station in cooperation with the party desiring the information.

Experience has shown that the wire gage is too inaccurate and temporary to be longer used by this Survey. Those now in use should be replaced as soon as possible by wooden ones if the conditions at the station will permit. If a wooden gage can not be used, then a chain one should be used. It is desirable that all these chain gages should be alike. To this end application should be made to the office of the chief engineer for all the necessary hardware, including weight, chain, pulley, lock, and key.

There should be two bench marks at each station—one on the bridge, from which the elevation of the water surface can be obtained with a steel tape; the other on the shore (preferably a copper bolt in rock),

above high water. The length of the chain gage should be measured at each visit to the station; also the distance from the bench mark to the water, and if the chain length is found to have changed, it should be corrected by the adjusting device at the upper end of the weight. Whenever a new gage is installed, care should be taken to have its 0 (zero) exactly at the elevation of the 0 of the old one, so that a correction to the gage heights will not be necessary.

The length of chain, elevation of each bench mark, elevation of water surface corresponding to gage reading when gage was installed, and all data necessary for checking the gage reading from the measurement of surface elevation with the steel tape should be written on the underside of the cover of the gage box with an indelible pencil, so that gage chain can be adjusted, if necessary, before measuring discharge.

The standard United States Geological Survey lock should be used on the gage box, so that no time will be lost in finding a key when the gage is visited.

INSTRUMENTS AND CAMP EQUIPMENT.

By E. M. Douglas.

In order to anticipate the needs of the average field party, and to provide a guide for district engineer and chiefs of field parties, correspondence has been had with a number of field men, and various persons have been consulted regarding the articles needed for the work.

There has been brought together an abstract list of instruments, giving the sizes, extras, names of makers, etc., which seemed best adapted for the work of the Survey in general. In this list is included every instrument thought necessary for work now in hand. There is also given a descriptive list of camp equipment. In preparing these lists suggestions have been received from Messrs. A. P. Davis, C. H. Fitch, J. B. Lippincott, A. L. Fellows, John T. Whistler, S. A. Aplin, E. G. Paul, and others.

INSTRUMENTS.

Transits.—Made by Berger & Sons, catalogue number, 1c. Has 2½-inch compass needle, 6½-inch horizontal circle, two double verniers, reading to 30 seconds; double set of figures reading from 0 to 360 in opposite directions, 5-inch vertical circle, graduated to minutes, all graduations being on solid silver; 4-inch level on telescope, object glass 6⅛-inch clear aperture, erecting eyepiece; stadia wires set to read 5 feet on rod 500 feet distant. Magnifying power, 24 diameters.

Telescopic alidades.—Made by Fauth & Co.; 18-inch ruler, beveled on left edge, no graduations; 4-inch aluminum column, telescope 12-inch focal length; object glass 1⅞-inch clear aperture; inverting eyepiece, with prism if requested; stadia wires set to cover 5 feet on rod 500 feet distant; vertical arc, 2⅝-inch radius, facing eyepiece end; flush vernier, reading to minutes; reversible striding level, each division repre-

senting one minute of arc; box compass, 4-inch needle, attached to alidade ruler. (Circular level, 1-inch diameter, for leveling plane table, etc., separate.)

Box compasses.—For attaching to plane-table boards, or for use separately, 4-inch needle, one-half-inch arc.

Wye levels.—Telescope 20 inches in length over all, made by W. & L. E. Gurley; level bubbles, 10 seconds of arc for each division; stadia wires reading 3 feet on rod 300 feet distant.

Wye levels for running short lines and setting gages, made by W. & L. E. Gurley; length of telescope, 15 inches; stadia wires, 3 to 300 feet.

Mountain transits.—Made by W. & L. E. Gurley; 4-inch needle; horizontal circle, 5½ inches in diameter, graduated on silver to single minutes, figured in both directions, 0° to 360°; vertical circle, 4½ inches in diameter, graduated to single minutes; level on telescope; packed in leather case with carrying straps; extension tripod or stiff-leg tripod, as desired.

Aneroids.—Diameter, 2¾ inches, pocket style; heights, 3,000, 5,000, 8,000, 10,000, and 15,000 feet; the 3,000 feet reads altitudes to 10 feet, the 5,000 to 20 feet, and the others to 50 feet.

Field glasses.—"Lemaire" make; jointed frames; 5 inches long when closed; 1¾-inch objective.

Price current meter.—Made by W. & L. E. Gurley, in two sizes. Small size, total length, 15 inches; outside diameter of revolving cup wheel, 5 inches. Large size, extreme length, 23 inches; diameter of revolving cup wheel, 6 inches. All meters equipped with 50 feet of double-conductor insulated wire and supplied with electric buzzer, battery and chemicals for replenishing same, and lead weights.

Bell odometers.—Registering miles and hundredths. In ordering, size of wagon wheel must be given.

Plane-table boards.—Size, 24 by 31 inches, 18 by 24 inches, and 15 by 15 inches, the latter having compass attached if desired. Waterproof wooden or leather cases furnished for large-sized boards; leather or canvas case for 15 by 15 inch boards.

Plane-table tripods.—Fitted with Johnson style heads; made by W. & L. E. Gurley; 48-inch split legs. All boards mentioned above supplied with plates to fit this tripod.

Rain gages.—Weather Bureau standards.

Leveling rods.—New York pattern; made by W. & L. E. Gurley; reading by verniers to thousandths of feet, each tenth figured; graduated on side for stadia reading; furnished with canvas covers and detached plumbing levels. These rods should be used for all primary leveling. Philadelphia pattern; to be used for secondary leveling; made by W. & L. E. Gurley; self-reading to hundredths of a foot; reading by verniers to thousandths of a foot; without plumbing level.

Tapes, steel.—From Lufkin Rule Company; length, 300 feet, 100 feet, 50 feet, and 6 feet; 300-foot tape to be used with spring balance and thermometer, latter to be standard make; tape graduated entire length in feet; last foot at each end subdivided into tenths. One-hundred-foot tape one-fourth inch in width; detachable handles; graduated entire length in feet, tenths, and hundredths. Fifty-foot tape, graduated on one side to feet, tenths, and hundredths; graduated on reverse side to links. Six-foot tape, graduated to inches and sixteenths; graduated on reverse side to millimeters.

Tapes, metallic.—Length, 50 feet and 25 feet; graduated in feet, tenths, and hundredths.

Traverse plane tables, etc.—Board 15 by 15 inches, with compass attached; used with light tripods, and folding sight alidade 6 inches in length.

Drawing instruments.—Furnished on contract from Interior Department supplies Specify kinds and sizes desired. Sets not furnished.

Bench-mark tablets.—Standard, aluminum.

Bench-mark posts.—Standard, iron, 4½ feet long, with brass caps.

Drawing paper.—Double-mounted for plane-table sheets; single-mounted paragon paper for traverse work and general drawing.

Scales, 1:45000, 1:90000, and special flat boxwood, with celluloid edges.

Flagpoles.—Round iron pipe, three-fourths inch in diameter, with steel shoes, 6 feet in length.

Locke hand levels.—Standard size.

Abney hand or slope level.—Arc reads to five minutes.

CAMP EQUIPMENT.

Book bags, of heavy ducking, leather bound, will be furnished from this office on requisition.

Water barrels.—Wine or whisky barrels are generally the only ones obtainable. These should be of a size proportionate to the needs of the camp.

Saddle or pack blankets.—Purchase the ordinary grade of coarse blankets; not bed blankets.

Mess boxes.—These should be made by a carpenter. The length should be equal to the width of the wagon box inside. They should have double covers, so that when the top cover is turned back the two covers will make a table. In size they should be in proportion to the camp, but not so large that when packed with supplies they will be unreasonably heavy to lift into the wagon.

Stationery boxes.—These may be made by a carpenter, or steamer trunks may be purchased. A stationery box designed by John T. Whistler is recommended.

Riding bridles.—These and similar articles should be plain, of good quality, but not with fancy fittings.

Buckets or pails should always be of galvanized iron; never of granite iron.

Canteens.—Covered, 3-pint army canteens will be supplied by the office on requisition. Larger canteens, of galvanized iron or tin, and holding from one-half to 1 gallon, may be purchased if necessary.

Chairs should be folding, all wood, with slats for seats (not perforated).

Stools should be folding, with heavy canvas seats.

Table cloths should be of oilcloth, cotton, or medium grade linen, white or colored.

Cups.—Cups and all dishes for use on table should be of white enameled ware, preferably with white outside. China, earthenware, glass, or tin should not be bought for use on camp tables. Gray granite ironware may be purchased if the white enameled ware is not available.

Flags.—A standard flag, 3 by 5 feet in size, and also the regular survey flag, should be displayed over every camp during working hours. These will be furnished from the office on requisition.

Tent floors.—Worn-out tents may be cut up and the canvas used for floors, or 12 or 14 ounce canvas may be purchased for the purpose. Wood floor of matched boards is preferred for a permanent camp. Floors should be provided for all sleeping and office tents.

Table forks, teaspoons, and tablespoons should be silver-plated on white metal.

Table knives should be silver-plated on steel.

Harness should be of good quality, but not fancy. Heavy styles, with collars, preferred.

Harness blinds should bear nickel-plated letters, U. S.

Branding irons for branding animals and numbers for hoof brands will be furnished from the office on requisition.

Kettles.—Cooking kettles should be of sheet iron or sheet steel and of different sizes, so that when packed they will nest one in the other.

Lamps.—Kerosene-oil hanging lamps should be provided for the cook's tent, and table or hanging Rochester lamps for each office tent; smaller sizes for each sleeping tent. "Electrolite" acetylene lamps may be purchased for use where transportation is by pack train.

Napkins, cotton or linen, white or colored, should be provided.

Bread and dish pans should be of tin only; never of granite iron.

Stewpans, stew basins, and similar dishes for use on the stove should be of granite iron.

Riding saddles should be plain, substantial, and without elaborate ornamental fittings.

Saws should be of medium length, with neither coarse nor fine teeth.

Stoves.—Cooking stoves should be of sheet or cast iron, or Donavin ranges may be used. Heating stoves may be of the Sibley style, but sheet iron or small cast-iron box stoves are preferred.

Tables should be of size proportionate to the number of men in camp, to be made by a carpenter, and should be folding or with detachable legs. The "Johnson" style is recommended. For a party of six or eight men a good size table is 6 feet long, 3 feet wide, 28 or 30 inches high. For the office tent a rolling table may be purchased if preferred to those made by the carpenter.

Tents.—Wall tents, 7 by 8, 9 by 9, or 14 by 14 feet, all but the first with flies, with or without poles or pins, can be procured on requisition from United States quartermaster, either in drab or khaki color. For office tents, white canvas is preferable and may be purchased from dealers. The 10 by 12 and 12 by 16 tents designed by A. L. Fellows are recommended in preference to the army tents. They may be secured from the Denver contractor through Mr. Fellows.

Wagons.—For small parties 2½-inch steel skein Studebaker wagons, and for larger parties one or more 3-inch steel skein Studebaker mountain wagons are recommended, of proper gage for the field of work. The 3-inch wagon should have high seat, and both sizes should have bows and canvas cover. Each should have on the brake side a box fitted for carrying a 5-gallon kerosene oil can. On the opposite side should be a stand for water barrel, above which iron hooks should be provided on which to carry tent poles and stovepipe. On the rear a rack, braced by a chain or rod, may be placed if desired, for carrying baled hay, stoves, etc.; a tool or "jockey box" should be in front. For light wagons to be used in transporting men and instruments, 1¼-inch spring wagons of standard make are preferred.

SURVEYS IN OKLAHOMA.

By Gerard H. Matthes.

On August 6, 1901, there was opened to settlement the southwest portion of Oklahoma Territory, previously held in reserve by the Federal Government for the Kiowa, Comanche, and Apache Indians. Three counties came thus simultaneously into existence, called Kiowa, Caddo, and Comanche counties, with county seats at Hobart, Anadarko, and Lawton, respectively. The lots in the town sites of these now so-called cities were sold by the Government to the highest bidder, the proceeds of the sales aggregating a sum of nearly three-quarters of a million dollars, and on these lands, in the incredibly short period of fourteen months, there arose cities with populations ranging from 3,300 to 8,000 inhabitants. Though at first mere agglomerations of hastily erected tents, these "rag towns," built out on the open prairie,

soon underwent complete metamorphoses, and in the course of two years assumed the aspects of towns. In keeping with the mushroom-like growth of these cities the adjacent country was rapidly taken up, and as claim after claim was fenced in the once boundless prairie was made to bristle with fence posts and barbed wire.

The act of March 3, 1901 (Stat. L., vol. 31, p. 1094), which opened the new country to settlement, provided, among other things, that the proceeds of the town-lot sales should be set aside for the construction of public improvements in the three counties and their county seats. The following paragraph is an extract from this law:

> The receipts from the sale of these lots in the respective county seats, shall, after deducting the expenses incident to the surveying, subdividing, platting, and selling of the same, be disposed of under the direction of the Secretary of the Interior in the following manner: A court-house shall be erected therewith at such county seat, at a cost of not exceeding ten thousand dollars, and the residue shall be applied to the construction of bridges, roads, and such other public improvements as the Secretary of the Interior shall deem appropriate, including the payment of all expenses actually necessary to the maintenance of the county government until the time for collecting county taxes in the calendar year next succeeding the time of the opening. No indebtedness of any character shall be contracted or incurred by any of said counties prior to the time for collecting county taxes in the calendar year next succeeding the opening, excepting where the same shall have been authorized by the Secretary of the Interior.

The act was amended on June 30, 1902 (Stat. L., vol. 32, pt. 1, p. 506), the amendment reading:

> *Provided further*, That the Secretary of the Interior be, and he hereby is, authorized, out of the proceeds of the sale of town lots in the towns of Lawton, Comanche County; Anadarko, Caddo County; and Hobart, Kiowa County, in the Territory of Oklahoma, heretofore had pursuant to the authority of the act aforesaid, to cause to be expended, subject to his control and supervision and upon the recommendation of the legally constituted authorities of each of said towns, for the construction of public waterworks, schoolhouses, and such other municipal improvements as may be advisable and advantageous to the inhabitants of said towns, the following additional sums, to wit:
>
> For the town of Lawton, one hundred and fifty thousand dollars, for the town of Anadarko, sixty thousand dollars, and for the town of Hobart, fifty thousand dollars: *Provided further*, that the sum of ten thousand dollars, provided in the act whereof this is amendatory for the construction of a county court-house in each of the towns aforesaid, shall be and hereby is increased to the sum of thirty thousand dollars each for the construction of such court-house in each town.

It is to be noted that this latter act requires a recommendation by the legally constituted authorities of each of the towns in the matter of waterworks, schoolhouses, and other municipal improvements. This is not, however, requisite in the case of county works.

Among the first public improvements to be made, bridges figure conspicuously. Efforts were also made by the various local authorities to provide for court-houses, jails, schools, and waterworks, but at this stage of the proceedings the Department in Washington found it neces-

sary to reject a number of plans and to annul contracts on account of the inefficiency and inadequacy of the improvements contemplated; and on account also of irregular and unbusinesslike proceedings on the part of the city and county officials. The failure of these officials to deal successfully with public matters of such paramount importance, and the serious apprehensions entertained by numerous citizens in the three counties, who realized that the large expenditure involved in the construction of these public works offered unusual opportunities for theft, resulted in a number of requests being directed to the Secretary of the Interior, asking that the Federal Government detail competent men to superintend the letting of contracts and to inspect their construction.

When, in March, 1903, engineers of the Geological Survey were detailed for this purpose, they were confronted with a condition of affairs which well-nigh baffle description. Months of careful investigation were required to straighten out the tangles and difficulties in which matters relating to public improvements had been allowed to become involved. It was found that such work as had been accomplished had in no instance been done in full compliance with the provisions of the Session Laws of Oklahoma or with such regulations as had been promulgated by the Secretary of the Interior relating to the manner of appropriating money for public improvements, to be paid for out of the town-lot fund.

In one county innumerable small bridges and culverts had been constructed without proper authority from the Secretary of the Interior. Examination of the claims submitted for the building of some of these bridges and culverts showed that not only had exorbitant prices been charged for lumber, but that more material had been charged for than was actually used. It also became known that bridge lumber had been purloined and used for private purposes for building dancing platforms and hay racks. Several bridge contracts had been let for steel structures without providing for proper approaches, so that the bridges, although built, remained inaccessible for travel for a long period. As one local critic expressed himself, "It required a balloon to get up on one end of some of these bridges, and a parachute to get down off the other end."

In a number of cases the bridges contracted for were too short to properly span the streams, and are, therefore, of doubtful or limited value, since they do not afford safe crossing at times of high water. In one instance the local authorities realized the fact that a bridge was too short for use at a chosen place and had it erected at a different point on the stream, overlooking the fact that the new site selected was on an Indian allotment and not on a public highway. The Indian agent, in order to stop travel over the Indian lands, had the approaches destroyed, and the bridge remains unused and unpaid for to this day,

and has gone down in the history of Comanche County as the "Lost bridge."

In Kiowa County contracts were let for the construction of a number of masonry bridge piers that were designed to support steel structures. It was found upon examination that in no instance had suitable foundations been provided; and, besides, the masonry was so poor that it had to be condemned as unfit for the purpose. Among the claims submitted to the Department for this work were found three claims for structures that had never been built. This county, which covers about 1,150 square miles, is still without highway bridges, except small wooden bridges and culverts. The Secretary ordered an investigation of the matter, and a number of indictments have been brought against county commissioners and contractors. In the city of Hobart the construction of a court-house was stopped, it being found that the foundations were worthless. The mortar used was so far from strong that it lacked even the quality of being adhesive.

It has been the duty of the engineers of the Geological Survey not only to investigate the conditions above described and to furnish to the United States attorney the evidence necessary for the prosecution of the guilty parties, but also to investigate the sources of water supply for the three county seats, to provide plans, specifications, and forms of contract, and to advertise for proposals for the construction of the various public works mentioned. In doing this work they are confronted almost daily with the difficulty of having to advise and confer with local officials, who in a majority of cases appear to have no conception of the duties that pertain to their respective offices. These officers are not disposed to cooperate with the Department, and it is difficult to get them to make proper recommendations to the Secretary for the construction of works that will be adequate for the purposes for which they are needed and reasonable in cost. After this difficulty is overcome, it is necessary finally to secure the sanction of the Secretary for the construction of the improvements desired; and where so many authorities have to be consulted much time is necessarily required, and progress under such conditions becomes slow. To further complicate matters, the amounts set aside from the town-lot funds are in many instances insufficient for the purposes for which they are intended, and the towns are required to supply the deficiencies by voting bonds.

During the summer season of 1903 an irrigation reconnaissance was carried on by members of the reclamation service in the western and northwestern counties of Oklahoma Territory, which are situated on the western edge of the semiarid belt, with a view to ascertaining the possibilities of constructing storage works and canals for irrigation. But two feasible reservoir schemes have been surveyed, though canal lines now cover large sections of land, mostly in Kiowa and Comanche

counties. The reconnaissance in the northwestern counties, which extended into the extreme western portion of Beaver County, resulted in the discovery of no irrigation prospects worthy of note. The soil in the neighborhood of large rivers is sandy, and any diversion of water from them will involve considerable loss by seepage.

The rivers of western Oklahoma, though they carry large amounts of water during the spring season, nearly all dry up in summer, and do not contain much water during the remainder of the year. Reservoirs can therefore be filled but once a year, and will be subject to great losses by evaporation, which it will not be possible to offset. In this part of the country irrigation is not absolutely essential to successful agriculture, but may be urgently needed once every two or three years. Fair crops were grown this year without the aid of irrigation.

Briefly, the outlook for irrigation enterprises in Oklahoma is not favorable. There are no vacant public lands under either of the reservoir sites surveyed, nor are there practical irrigators in that part of Oklahoma. Farmers here and there say they desire to have irrigation works built by the Government, but most of them seem to regard irrigation as a matter of experiment, and it is doubtful whether in the end they will be willing to pay for the costly storage works contemplated.

COLORADO RIVER RECLAMATION PROJECTS.

By E. T. PERKINS.

A consideration of the results of the investigation made in the winter of 1902–3 along Colorado River, from latitude 35° 12′ north to the Mexican line has convinced the engineers in charge that the following projects are either feasible or worthy of further examination. In order of their importance and urgent need of investigation the projects are ranked as follows:

First. A dam at the Yuma dam site, with canals from this point to lands in California and lands in Arizona.

Second. A diversion at the proposed headworks of the Blythe Estate canal in California.

(A strong argument for these two propositions is that California will thereby obtain its rights under the reclamation act.)

Third. A diversion at Head Gate Rock, above Parker, Ariz., with canal covering the Colorado River Indian Reservation in Arizona.

Fourth. A dam at Williams for storage purposes.

Fifth. A diversion at Bulls Head, Ariz., with canal leading to the lands of the Mohave Valley, Arizona. Also a further investigation for bed rock at Bulls Head for a high dam for storage purposes.

It is believed that storage sites exist at points above Bulls Head, but they have not been investigated.

Commencing at the upper project, at Bulls Head, and proceeding downstream, the various schemes under consideration present the fol-

lowing salient features. In an investigation for the Bulls Head dam, the first site proposed was abandoned. Back of the rocky, river hills there was exposed a contact point of clay and sand at an elevation sufficiently low to jeopardize the plan for a 100-foot dam at this place. Such a structure here would create a reservoir with a capacity of 845,000 acre-feet and a surface area of 16,750 acres. While the conditions were unfavorable for a dam so high, there is a possibility that a dam to be carried 60 or 70 feet above low water may be constructed, provided bed rock is sufficiently close to the surface. A dam 70 feet high would form a reservoir having a capacity of 392,000 acre-feet and a surface area of 13,600 acres. In consideration of this fact it is deemed necessary to continue the investigations here during the coming season. The capacity and surface acreage are computed only to latitude 35° 12′. We have no topographic maps beyond this latitude, but the river here enters a canyon, and it is not believed that much additional capacity exists above this.

DIVERSION AT BULLS HEAD.

Though no suitable place for a high dam has yet been found at Bulls Head it is feasible to construct here a diverting weir about 28 feet high above low water. From this weir could be carried a canal, which would cover 47,000 acres of land in Arizona between Bulls Head and Mellon. It is also possible, by diverting the river to a former course, to transfer to the Arizona side 10,600 acres which are now in California and Nevada, making a total acreage of 57,600 acres gross under the proposed canal. Of this area 36,000 acres were classified by surveying parties of the reclamation service as first and second class farming lands.

DAM AT WILLIAMS.

The topographic maps show that a 75-foot dam at the Williams dam site would create a reservoir with a capacity of 1,300,000 acre-feet and a surface acreage of 50,873 acres. Such a dam would not flood the town of Needles, nor would it compel the raising of much of the track of the Santa Fe Pacific Railroad, but it would probably necessitate the raising of the Red Rock Bridge at Mellon about 10 feet. This reservoir would probably be the largest of its class in the world, and would serve a good purpose in regulating the river and in silting the bottom land of the lower Mohave Valley. This proposition should be further investigated during the coming season.

DIVERSION AT HEAD GATE ROCK.

Below Head Gate Rock, on the Arizona side of the river, lie the low lands of the Colorado River Indian Reservation, comprising 129,000

acres. Two propositions for reclaiming this land present themselves. A canal might possibly be constructed from the Williams dam. Should this prove too expensive, a diversion may be made at Head Gate Rock. Previously such a diversion was attempted by the Indian Bureau, but bad engineering resulted in a failure. Such a canal would cover 129,000 acres above Ehrenberg, and probably about 100,000 acres below. Topographic maps of the region below Ehrenberg have not been completed and we can not compute accurately this lower acreage. Almost this entire area is in Government possession. The length of canal from Williams dam to Head Gate Rock would be 15 miles. A high-line canal at this point would not reclaim materially more valuable land than would be reclaimed by a low-line canal.

DIVERSION AT BLYTHE ESTATE.

On the California side, 10 or 12 miles above Ehrenberg, Ariz., is a possible diversion point. A canal from this point would cover the Blythe estate (40,000 acres) and adjacent lands, in all about 95,000 acres. It is possible that this canal could also be extended to cover Milpetas Valley, reclaiming altogether, as estimated by Mr. Marvin, supervisor of Yuma County, Ariz., 150,000 acres. This project is worthy of prompt consideration, as it is one of the very few large ones now known in California that seem worthy of serious consideration under the reclamation act.

PICACHO DAM SITE (OR GOLDEN DREAM DAM SITE).

At one time the construction of a dam at the Golden Dream dam site, for storage purposes, was favorably considered, but the present belief of the engineers in charge is that such a reservoir would injure more good land above the dam than could be additionally reclaimed below. For the present, therefore, the site is abandoned pending investigations below.

YUMA DAM SITE.

At a point 22 miles above Yuma, Ariz., the Colorado River passes through its last rocky gorge, the walls of which are about 1,500 feet apart. The topographic maps show that the construction here of a diversion dam 60 feet in height would enable the reclamation of 155,000 acres in Arizona and 23,000 acres in California in the river bottom, with a possibility of so extending the California canal as to cover all the lands of the Salton Sink, where it is roughly estimated there are about 300,000 acres of good land not now under ditch. This project is the only one so far discovered in which a high-line canal is considered

advisable. Of the Arizona acreage, 65,000 acres are mesa lands, which could be devoted to citrus fruits, the remaining 90,000 acres being bottom lands, suitable for general agricultural products.

The general plan for the reclamation of lands adjoining Colorado River, as proposed by Mr. A. P. Davis, supervising engineer of the reclamation service, provides that the project farthest down the river shall be first taken in hand, diversions to be made and reservoirs formed as needed, the dams being of such heights as to permit the silting of low bottoms which could afterwards be made agricultural lands by diverting water from lower outlets in the dam. The building of the Yuma dam would be in harmony with such a plan, and that project would also have independent intrinsic merit of the highest order. The extreme upper waters of the reservoir would cover the low lands lying between Picacho and Ehrenberg, but would not "drown" any large amount of land at present suited for agriculture. The desirability of extending the work in the way proposed by Mr. Davis, or in some similar way, would depend upon further study. The building of this dam is dependent upon the finding of bed rock at the dam site within reasonable depths.

This project is conveniently located for transportation facilities and would reclaim lands that now have a growing market for their produce. Furthermore, the construction of the system here proposed would aid a most deserving number of citizens of the United States, the farmers below Yuma, who are now struggling with defective irrigation and reclamation systems. These men lately obtained the titles to their land by brave and unwearying effort in the courts in conflict with claimants under a Spanish grant. They are pioneers in the settlement of a great region, and are undergoing all the deprivations and hardships which fall to such a lot. They are most worthy of aid at the hand of the Federal Government.

CONCLUSION.

The merits of the various propositions having been carefully weighed, with due regard to a general scheme of reclamation along the entire river, after consultation with the consulting engineers of the reclamation service it has been decided that all possible effort be made to push the Yuma dam site project to successful completion.

SUMMARY.

The following gross amounts of land would be reclaimed by the various propositions:

	Acres.
Diversion at Bulls Head	58,000
Diversion at Parker	230,000
Diversion at Blythe estate	150,000

	Acres.	Acres.
Diversion at Yuma:		
In Arizona	155,000	
In California	23,000	
In Salton Sink, approximately	300,000	
		478,000
Total		916,000

RECLAMATION AND WATER STORAGE IN NEBRASKA.

By O. V. P. Stout.

AGRICULTURAL CONDITIONS.

The position of Nebraska as an irrigation State is unique. The eastern part lies well within the region where the normal precipitation, in amount and in seasonal distribution, is favorable to liberal yields of general crops. The western part of the State, on the other hand, is favored with sufficient rainfall for successful agriculture in occasional years only, or in short series of years. The change from the one condition to the other, as we pass over the intervening territory, is gradual. For more than 200 miles west of Missouri River lands are cultivated by men who have risen from poverty to affluence through the practice of agriculture without irrigation, and until we go still farther west we find that the burden rests too heavily upon irrigation enterprises to justify their existence and to repay the money which has been invested in them. Even in the extreme western part of the State, where to attain even reasonably uniform success in raising crops the aid of irrigation must be invoked, the first advocates and promoters of irrigation met with determined resistance on the part of those whose real or fancied interest lay in opposition to the conception of that country as an arid or semiarid region. Since about 1890, however, irrigation development has enlisted the eager attention and active effort of practically all persons whose homes or investments lay in that locality.

A very large proportion of the area of Nebraska consists of tillable land. The only deduction of consequence to be made is the sand-hill region of the northwest-central portion of the State. Thus it is that while it may be possible to irrigate in Nebraska a percentage of the total area approaching that which obtains in the States which have extensive mountain areas, it is frequently made to seem smaller through comparison with the tillable areas instead of the total areas of the States.

Practically nothing has yet been done to conserve the surplus and unseasonable water supply, and much remains to be done to perfect methods of conveyance, distribution, and use, to the end that the greatest possible area may be served by the available water.

The period of irrigation development thus far has been also a period of general readjustment in the portions of the State which are most directly affected. The high dry plains, at one time cultivated, have reverted to a large extent to grazing. Irrigation has been introduced in the valleys. Alfalfa has become a leading crop. The corn crop, once preeminent, has partly given way to small grain. It has been demonstrated that sugar beets make a successful crop in many localities. The value and adaptability of kaffir corn has been discovered. Some of the most experienced fruit growers claim to have demonstrated that certain localities in the semiarid country are especially adapted to their industry. The winter feeding of range stock has become general, and this and other demands of the stockman furnish a large market close at hand for the surplus products of the irrigated land.

The present tendencies are toward continued irrigation development, in the nature of better use of the facilities which have been provided and the further provision of facilities for the conservation and use of the water which now runs to waste. More attention will be given to markets, with a view to supplying their demands. The prospective industrial development of Colorado and Wyoming will create new markets which irrigated Nebraska will have a share in supplying.

NORTH PLATTE RIVER.

The principal source of water supply for irrigation in Nebraska is North Platte River. Although the greatest number of canals, the largest canals, and the greatest irrigated acreage in the State are in this valley, only a relatively small portion of the water which the stream carries is used. It is true that the season of low water, from late July into September, is frequently a time of apprehension on the part of irrigators, but the very large excess of the flood volume, and practically the entire winter flow of several thousand cubic feet per second, run to waste.

The drainage area contributing to the flow at Mitchell, Nebr., about 15 miles east of the Wyoming line, is 24,400 square miles, which includes all of the mountainous portion of the drainage area.

The United States Geological Survey has maintained gaging stations at various points on the river since 1895. A study of the resulting records has been made with a view to determining the proportion of the total supply which different volumes of storage would make available. In the absence of definite figures on a number of the points involved, it has been found necessary to assume figures which seem reasonable and probable. It has been found impracticable to obtain continuous winter records of the flow of the stream without resort to unusual and unduly expensive methods. Consequently the discharges

for the months which were lacking in the record were interpolated. The demand was assumed to cover all use and all losses from canals and reservoirs. Two separate assumptions were made in respect to this demand. In the first, designated as demand A, it was assumed that November, December, January, February, and March would each require 10 per cent of the amount required for June; April and September would each require 50 per cent; May, 80 per cent; July, 100 per cent; August, 85 per cent, and October, 25 per cent. For demand B, the demand for the months of the year in succession were respectively 5, 5, 5, 20, 80, 100, 110, 90, 30, 15, 5, and 5. Then calculations were made as to the amount of storage necessary to make available different percentages of the total supply, ranging from 60 to 100 per cent. It was assumed that stored water would be drawn upon only when the flow in the stream was unequal to the demand. The years 1895 to 1902 were embraced in the investigation. The following table sets forth the supplying capacity of the river, as ascertained and estimated from the records:

Discharge of North Platte River in thousands of acre-feet.

Year.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1895	123	139	185	203	433	644	198	28	14	49	81	92
1896	123	115	123	168	273	377	95	57	56	62	89	92
1897	123	111	123	121	859	827	125	65	29	30	128	123
1898	123	111	123	97	274	418	100	16	7	15	62	184
1899	185	167	308	396	630	954	752	174	64	84	108	197
1900	154	173	215	322	640	581	127	30	17	23	30	92
1901	123	139	145	143	472	447	145	25	19	24	29	92
1902	92	111	139	95	267	292	80	13	.4	18	34	62

From Table I, page 280, based on the above mean monthly flow of the river near the Wyoming-Nebraska State line, the storage necessary each month and year to utilize 60 per cent of the total supply on the basis of demand B may be ascertained. All the quantities except those in the first line, "Demand in per cent," are given in thousand acre-feet. The minus sign indicates a deficiency of water flowing in the river, the flow is not sufficient to satisfy the demand, and hence storage is necessary. All the other numbers signify an excess of water after the monthly demand is satisfied.

These numbers are obtained as follows: The total supply for the eight years is 17,437,400 acre-feet. The average annual supply is one-eighth of this amount, which equals 2,179,675 acre-feet. Sixty per cent of this equals 1,307,805 acre-feet. The demand for the year, expressed in per cent, is 470 per cent of the June demand, as seen from the first line of the table. The June demand in acre-feet is $1,307,805 \div 4.70 = 278.26$ thousand acre-feet. The monthly demand for each of the other months is given in the second line of this table and is found by multiplying the June demand by the per cent for that month. The difference between the monthly flow and the monthly

demand is the excess or deficiency for that month, according as the flow is greater or less than the demand. Take for example the year 1896. There is a deficiency in July of 211.10 thousand acre-feet and storage of this volume must be provided for this month. There is also a deficiency in August and September of this year amounting to 193.43 and 27.48 thousand acre-feet, respectively. The total deficiency for the year is 432.01 thousand acre-feet, and since the period of deficiency is continuous this is the volume of storage necessary for the year. The excess between the first of the year and the beginning of the period of deficiency is 437.62 thousand acre-feet, and being larger than the deficiency it would not be necessary to store any water until the first of the year. In 1902, however, it is seen that the deficiency is greater than the accumulated excess since the close of the period of deficiency in 1901, and consequently storage equal in amount to 11.33 thousand acre-feet must be carried over from June of 1901.

Table II sets forth the corresponding figures when it is assumed that the entire supply will be utilized on the basis of demand A. It is seen that the greatest deficiency for any one year, considered by itself, occurs in 1902, and amounts to 1,218.81 thousand acre-feet. To find the volume of storage necessary in order that there may be a sufficient amount of water at all times to meet the demand, and at the same time no water shall be permitted to run to waste, the monthly excesses and deficiencies must be combined by months throughout the entire period of eight years. This has been done in the third of the following tables, in which each tabular quantity is the algebraic sum of all corresponding quantities in the second table, together with the addition throughout of 1,000 thousand acre-feet, in order to avoid negative quantities. The addition of this 1,000 thousand acre-feet is equivalent to assuming that stored water to the extent of that amount was available at the beginning of the period under consideration. It is found that the lowest stage of the stored water occurs in October, 1898, and the highest in June, 1900. The range between these two extremes is $3{,}164.23 - 124.08 = 3{,}040.15$ thousand acre-feet, and this is the capacity of the reservoir or reservoirs which will fulfill the conditions which have been imposed, namely: No waste and no unsatisfied deficiency.

From the first of the following tables it appears that on the basis of demand B, using 60 per cent of the total supply and permitting 40 per cent to be wasted, storage to the extent of 570,350 acre-feet would be required. If reservoir area equal to one township be assumed, these figures correspond to an average depth of 22 feet. Similarly, to utilize the entire supply, averaging 2,179,675 acre-feet yearly, reservoir capacities equivalent to about 132 feet in depth over a township would be required. The capacity of such reservoirs would be sufficient to hold nearly 50 per cent more than the average discharge for one year.

TABLE I.—*Monthly increment of excess or deficiency, assuming the utilization of 60 per cent of the entire supply from 1895 to 1902, on the basis of demand B.*

Year.	January.	February.	March.	April.	May.	June.	July.
Demand in per cent[a]	5	5	5	20	80	100	110
Demand, acre-feet	13.91	13.91	13.91	55.65	222.60	278.26	306.10
1895	109.09	125.09	171.09	147.35	210.40	365.74	−113.10
1896	109.09	101.09	109.09	112.35	50.40	98.74	−211.10
1897	109.09	97.09	109.09	65.35	636.40	548.74	−181.10
1898	109.09	97.09	109.09	41.35	51.40	139.74	−206.10
1899	171.09	153.09	294.09	340.35	407.40	675.74	+445.90
1900	140.09	159.09	201.09	266.35	417.40	302.74	−179.10
1901	109.09	125.09	131.09	87.35	249.40	168.74	−161.10
1902	78.09	97.09	125.09	39 35	44.40	13.74	−226.10
Total	934.72	954.72	1,249.72	1,099.80	2,067.20	2,313.92	−831.80

Year.	August.	September.	October.	November.	December.	Total yearly excess.	Total yearly deficiency.	Available for additional storage.
Demand in per cent[a]	90	30	15	5	5	.....	.....	.....
Demand, acre-feet	250.43	83.48	41.74	13.91	13.91	.....	.....	.....
1895	− 222.43	− 69.48	+ 7.26	67.09	78.09	1,281.20	405.01	+ 876.19
1896	− 193.43	− 27.48	+20.26	75.09	78.09	754.20	432.01	+ 322.19
1897	− 185.43	− 54.48	−11.74	114.09	109.09	1,788.91	432.75	+1,356.19
1898	− 231.43	− 76.48	−26.74	48.09	170.09	765.94	543.75	+ 222.19
1899	− 76.43	− 19.48	+42.26	94.09	183.09	2,807.10	95.91	+2,711.19
1900	− 220.43	− 66.48	−18.74	16.09	78.09	1,580.94	484.75	+1,096.19
1901	− 225.43	− 64.48	−17.74	15.09	78.09	963.94	468.75	+ 495.19
1902	− 237.43	− 83.08	−23.74	20.09	48.09	465.94	570.35	− 104.41
Total	−1,595.44	−461.44	−28.92	449.72	822.72	10,408.20	3,433.18	+6,974.92

TABLE II.—*Monthly increments of excess or deficiency, assuming the utilization of the entire supply from 1895 to 1902, on the basis of demand A.*

Year.	January.	February.	March.	April.	May.	June.	July.
Demand in per cent[a]	10	10	10	50	80	100	100
Demand, acre-feet	40.36	40.36	40.36	201.82	322.92	403.64	403.64
1895	82.63	98.64	144.64	+ 1.18	110.08	240.36	− 210.64
1896	82.63	74.64	82.64	− 33.82	− 49.92	− 26.64	− 308.64
1897	82.63	70.64	82.64	− 80.82	536.08	423.36	− 278.64
1898	82.63	70.64	82.64	−104.82	− 48.92	14.36	− 303.64
1899	144.63	126.64	267.64	194.18	307.08	550.36	+ 348.36
1900	113.63	132.64	174.64	120.18	317.08	177.36	− 276.64
1901	82.63	98.64	104.64	− 58.82	149.08	43.36	− 258.64
1902	51.63	70.64	98.64	−106.82	− 55.92	− 111.64	− 323.64
Total	723.04	743.12	1,038.12	− 69.56	+1,264.64	+1,310.88	−1,612.12

Year.	August.	September.	October.	November.	December.	Total yearly excess.	Total yearly deficiency.	Available for additional storage.
Demand in per cent[a]	85	50	25	10	10	.....	.....	.....
Demand, acre-feet	343.10	201.82	100.91	40.36	40.36	.....	.....	.....
1895	− 315.10	− 187.82	− 51.91	40.64	51.64	769.81	785.47	.....
1896	− 286.10	− 145.82	− 38.91	48.64	51.64	340.19	889.85	.....
1897	− 278.10	− 172.82	− 70.91	87.64	82.64	1,365.63	881.29	.....
1898	− 327.10	− 194.82	− 85.91	21.64	143.64	415.56	1,065.21	.....
1899	− 169.10	− 137.82	− 16.91	67.64	156.64	2,163.17	323.83	.....
1900	− 313.10	− 184.82	− 77.91	− 10.36	51.64	1,087.17	862.83	.....
1901	− 318.10	− 182.82	− 76.91	− 11.36	51.64	529.99	906.65	.....
1902	− 330.10	− 201.42	− 82.91	− 6.36	21.64	242.55	1,218.81	.....
Total	−2,336.80	−1,408.16	−502.28	+238.12	+611.12	6,914.06	6,913.94	.....

[a] All other figures are in thousand acre-feet.

TABLE III.—*Amount of water in reservoirs at end of each month, assuming that 1,000 acre-feet are stored at the beginning of the period considered (Demand A, entire supply utilized).*

Year.	January.	February.	March.	April.	May.	June.
1895	1,082.63	1,181.27	1,325.91	1,327.09	1,437.17	1,677.53
1896	1,086.97	1,161.61	1,244.25	1,210.43	1,160.51	1,133.87
1897	537.31	607.95	690.59	609.77	1,145.85	1,569.21
1898	1,021.65	1,092.29	1,174.93	1,070.11	1,021.19	1,035.55
1899	433.99	560.63	828.27	1,022.45	1,320.53	1,879.89
1900	2,242.33	2,371.97	2,549.61	2,669.79	2,986.87	3,164.23
1901	2,435.67	2,534.31	2,638.95	2,580.13	2,729.21	2,772.57
1902	2,028.01	2,098.65	2,197.29	2,090.47	2,034.55	1,922.91

Year.	July.	August.	September.	October.	November.	December.
1895	1,466.89	1,151.79	963.97	912.06	952.70	1,004.34
1896	825.23	539.13	393.31	354.40	403.04	454.68
1897	1,290.57	1,012.47	839.65	768.74	856.38	939.02
1898	731.91	404.81	209.99	124.08	145.72	289.36
1899	2,228.25	2,059.15	1,921.33	1,904.42	1,972.06	2,128.70
1900	2,887.59	2,574.49	2,389.67	2,311.76	2,301.40	2,353.04
1901	2,513.93	2,195.83	2,013.01	1,936.10	1,921.74	1,976.38
1902	1,599.27	1,269.17	1,067.75	984.84	978.48	1,000.12

Estimates and computations, of which the foregoing are examples, have resulted in the figures set forth in the following table:

Estimated necessary storage of waters of North Platte River entering Nebraska.

[Unit of volume of storage is 1,000 acre-feet.]

Percentage of supply utilized.	Demand A.	Demand B.
60	518.00	570.35
70	817.00	885.00
80	1,100.00	1,152.40
90	1,615.00	1,662.34
95	2,333.69	2,416.00
100	3,040.15	3,126.60

According to the best estimates now obtainable relating to the duty of water in Nebraska, the total flow of the river, if made available, would suffice for the complete irrigation of 1,000,000 acres. The State engineer of Nebraska reports that 560,000 acres are now under ditch in the Platte and North Platte valleys. It is known, however, that not nearly all of this is irrigated.

No reservoir sites of significant magnitude, presenting favorable conditions for construction, and situated to serve for the storage of the waters of the North Platte River, have as yet been discovered in Nebraska. Examination of the topographical maps published by the United States Geological Survey indicates that sites suitably situated and in other respects fairly favorable to the service of Nebraska land, may be found in Wyoming. A reconnaissance by the State engineer of Nebraska has verified these indications to a considerable extent. The work necessary to take advantage of these sites would be of great magnitude and would cost a vast sum of money. Since the largest

reservoir sites lie at considerable heights above the river the large feeder canals would have great length. It has been suggested that the complete utilization of North Platte waters can be most successfully accomplished by diverting the entire flow at all times into a canal which will lie at a greater elevation than the reservoir sites which may be determined upon as available, and will serve as a feeder to all of the reservoirs. As a part of this scheme it is contemplated that all present or future canals shall derive their supply from this main canal or from the reservoirs which are fed by it. A necessary and economical modification of the scheme is to the effect that a portion, at least, of the waters of maximum floods should be diverted from the river into the minor canals, as it would be extravagant to construct the main canal of size to carry the entire volume of these maximum floods. A canal constructed of such size would be utilized to its full capacity only in the most exceptional years. It is doubtful if it would be wise to approach this size of main canal so nearly that the excess of maximum floods would all be taken from the river by the minor canals. In other words, until water has a value for irrigation considerably in excess of what may be expected in this generation in the North Platte Valley it will not be advisable to undertake to make available the entire discharge of the stream and to prevent all of the waste which ensues from the passage of the water down the river.

The considerable reservoir sites in Wyoming in which water may be stored for Nebraska lands lie on the north side of the river, so far as they have come to the attention of the writer. When we come to consider the high lands—that is, the lands higher than first and second bottom—it is found that a vastly larger acreage is susceptible of irrigation on the south side of the river than on the north. The advantages and disadvantages of each site as they shall be brought to light by surveys and other investigations should be carefully considered, in order to decide on which side of the river the large development shall be undertaken or whether the large potential supply of water shall be divided between the two sides.

OTHER RIVERS.

On the upper part of Republican River, and on its tributary, Frenchman River, agriculture without irrigation is an occupation yielding uncertain results. At the same time agriculture with irrigation meets with satisfactory returns. Hence the irrigation ditches which have been constructed are used regularly to the extent of the supply of water available from the streams, and better and better results are obtained by the irrigators from year to year. The problem of storing and utilizing the supply, which under present conditions is not available, is now an important one in the minds of the users of water, and

it is safe to assume that some attention will be given to it by the engineers of the reclamation service in the near future. The conditions are such that little can be done toward the solution of the problem until it has been taken up on the ground in detail. Since no part of the drainage area is mountainous, the floods are due entirely to rainfall, which is mainly of irregular occurrence in point of time and of uncertain magnitude.

The unseasonable discharge which at present runs to waste would be the principal and most definite item to be considered. It may be estimated roughly that if this could be conserved the irrigating capacity of the streams would be doubled. This would be possible only in case sites suitable for the storage of 75,000 to 100,000 acre-feet can be found. No surveys or examinations other than the most casual have been made to determine if such sites can be located.

Reckoned on the basis of total discharge, Loup River is second to the North Platte. The seasonal distribution of its discharge is, however, markedly different. The discharge arises mainly from the precipitation in the sand-hill region, which is of such a character that even the heaviest rains sink at once and almost entirely into the soil and the water reaches the streams in the form of springs, resulting in a flow which is subject to only relatively small variations. The total discharge of the Loup at the gaging station a few miles above its mouth, during the irrigation season of the year, is about three-fourths that of the North Platte for the same period. Even in the case of a discharge presenting such a marked degree of uniformity it can not of course be assumed that the demand and the supply will be exactly conformable to each other. If, therefore, irrigation from the stream is ever developed to such a point that the supply is at times unequal to the demand upon it, the question of storage will have to be considered. There is a prospect that practically the entire flow of the river will be used at an early date for the development of power at some point in its lower course or after the water has entered the bed of Platte River. If projects now in contemplation are carried to completion and their rights to the use of the water are perfected, no storage except that of the surplus waters of occasional floods can be accomplished without considering these rights. This would complicate the problem of the storage of the winter flow. It is not known that any consideration has been given to the problem of extensive water conservation on this stream. The character of the stream in respect to the extent and uniformity of discharge and the relatively small amount of irrigation development which has taken place on the stream and its tributaries have combined to make the storage of its waters seem less important than that of the waters of some of the other streams. It may be expected that when an investigation along this line is taken up on

Loup River there will be some difficulty in finding reservoir sites of sufficient capacity, properly situated, and attended by suitable geological conditions.

It is not believed that Niobrara River will come early into prominence as an irrigation stream. Its course through the semiarid region lies for the most part in the sand hills, a region which presents very limited prospects for general agriculture. Moreover, the stream runs at a great depth (more than 100 feet in many places) below the land which it might be desirable to water, so that long lines of diversion canals would be necessary to lead the water to the land. As these canals would have to be constructed for the most part in soil of decidedly unfavorable character, so far as the capacity to hold water is concerned, it is probable that their efficiency would be small and that great losses and waste would ensue. Aside from a considerable number of minor irrigation enterprises on the upper portion of the river, it has been considered mainly as a source of supply of water for the production of power. At least two large projects have been receiving the attention of promoters for a number of years, and it seems that the natural advantages in respect to topography and water supply are such that, as soon as assurance can be given that there will be a demand for the power which may be generated, the plants will be installed.

The natural advantages in respect to water supply are the considerable amount of the discharge and the almost unequaled degree of uniformity which attaches to it. The lowest measured or estimated discharge of the stream is about 600 cubic feet per second. The highest is about 1,500 cubic feet per second. The ratio of maximum to minimum observed discharge is therefore only two and one-half. This exceedingly low ratio is a guaranty of its reliability as a source of supply for water power, and so far as its utilization for that purpose is concerned, there is no occasion to take up the discussion of the artificial regulation of the flow by storage or otherwise.

In addition to the principal streams which have been discussed there are a number of minor streams which are rendering good service in irrigation, or have the capacity to render such service. Some of these, also, it may be well to investigate at some time, with a view to determining the extent to which their usefulness can be increased by storage and other methods of conservation.

PORTLAND-CEMENT MANUFACTURE.[a]

By Edwin C. Eckel.

The important building material known as Portland cement was first manufactured in England in 1824. Its name is not, as is commonly supposed, due to the fact that it is manufactured at Portland

[a] Revised from a series of articles published in Municipal Engineering during 1902.

in England, Portland in Maine, or Portland in Oregon, for it is not and never has been made at these points. The name "Portland" was applied to this cement because, after setting, it resembles somewhat a gray limestone extensively quarried at Portland, England. The original patents on Portland cement expired long ago, and it may now be manufactured at any point where suitable raw materials are obtainable.

As defined by good authorities, Portland cement is an artificial product, obtained by burning to semifusion an intimate mixture of finely pulverized materials, this mixture containing lime, silica, and alumina in certain fixed and definite proportions, after which burning the resulting "clinker" is crushed very finely. Four points may therefore be regarded as of cardinal importance in the manufacture of Portland cement. These are:

1. The cement mixture must be of the proper chemical and physical composition.
2. The materials of which it is composed must be finely ground and intimately mixed before burning.
3. The burning must be conducted at the proper temperature.
4. After burning, the resulting clinker must be finely ground.

These points will be taken up in order.

RAW MATERIALS.

For the present purposes it is sufficiently accurate to say that a Portland-cement mixture ready for burning contains approximately 75 per cent lime carbonate and 20 per cent clayey material, i. e., silica, alumina, and iron. The essential elements—lime, silica, alumina, and iron—entering into this mixture are very common and widely distributed in nature, and it is obvious that such a mixture could be made by combining in an almost infinite number of ways many different raw materials. The necessity for producing the mixture as cheaply as possible, however, rules out a number of raw materials which would be available so far as their composition only is concerned. In the following pages the combinations of raw materials now actually used in the United States in the manufacture of Portland cement are grouped under six heads. Each of these six different combinations of raw materials will be treated separately, brief descriptions of their chemical composition, economic advantages, and geographic distribution being given.

Argillaceous limestone and pure limestone.—An argillaceous limestone containing approximately 75 per cent lime carbonate and 20 per cent of clayey materials would, of course, be the ideal Portland-cement material, as it would require no addition of other material in order to give, on burning, a good Portland cement. Certain deposits of argillaceous limestone occurring in the United States approach this ideal composition very closely. The most important of these deposits at

present is that which is so extensively utilized for Portland cement in New Jersey and Pennsylvania. As this district still produces about two-thirds of all the Portland cement made in the United States, its raw material will be described in detail.

The cement rock of the Lehigh, Nazareth, and Easton districts is a highly argillaceous limestone of Trenton (Lower Silurian) age. The rock is a very dark gray in color, with usually a slaty fracture. In composition it varies from 60 per cent lime carbonate with 40 per cent silica and alumina, up to 80 per cent lime carbonate with 20 per cent silica and alumina. Its magnesia content is always high, as Portland-cement materials go, varying from 3 to 6 per cent.

Underlying these cement beds are beds of purer limestone, ranging from 85 to 96 per cent lime carbonate. The usual practice in Pennsylvania and New Jersey plants is, therefore, to mix a relatively small amount of this purer limestone with the argillaceous "cement rock," and thus obtain a cement mixture of proper composition.

The technologic and economic advantages of using this combination of materials are well marked. Both the materials, particularly the "cement rock," are quarried easily and cheaply. As quarried they carry little water, so that the expense of drying them is slight. The fact that four-fifths or more of the mixture is made up of a natural cement rock permits coarser grinding of the mixture than is common in plants using marl or limestone and clay, and it seems probable that this fact also decreases the amount of fuel necessary to clinker the mixture.

Such mixtures of argillaceous limestone and pure limestone possess, therefore, important advantages over mixtures of limestone with clay or marl with clay, but are, on the other hand, more expensive than the combination discussed under "Chalky limestone and clay."

The analyses in the table below are fairly representative of the materials used in the Lehigh and adjoining districts, the first four being of "cement rock," the last two of pure limestone.

Cement rock and limestone.

	Cement rock.				Limestone.	
Silica	10.02	9.52	14.52	16.10	3.02	1.98
Alumina, Iron oxide	6.26	4.72	6.52	2.20	1.90	.70
Lime carbonate	78.65	80.71	73.52	76.23	92.05	95.19
Magnesium carbonate	4.71	4.92	4.69	3.54	3.04	2.03

Marl and clay.—In the northern and central Western States large deposits of calcareous marls occur, usually filling old lake basins and being due to the precipitation of lime carbonate from solution by the action of vegetable and animal life. These marls have formed the basis for a large Portland-cement industry and are now worked

extensively for this purpose, principally in Michigan, northern Indiana and Ohio, and central New York.

As shown by the analyses below, marls are usually very pure lime carbonates. They therefore require the addition of considerable clay to bring them up to the proper composition for a Portland-cement mixture.

The marls are readily excavated, but necessarily carry a large percentage of water. The mixture, on this account, is commonly made in the wet way, resulting in a very large consumption of fuel in the kilns. Analyses of typical marls and clays are given in the following table.

Marl and clay.

	Marl.			Clay.		
Silica	0.25	3.0	1.60	40.48	52.0	63.75
Alumina	.10		1.55	20.95	17.0	16.40
Iron oxide					5.0	6.35
Lime carbonate	94.39	93.0	88.9	25.80	20.0	4.0
Magnesium carbonate	.38	1.5	.94	.99		2.1

Chalky limestone and clay.—In certain Southern and Western States a chalky limestone is used in combination with clay. These chalky materials are geologically nearly all of Cretaceous age. The principal deposits available for use in Portland-cement manufacture occur in three distinct and widely separated areas, occupying, respectively, (*a*) parts of Texas and Arkansas, (*b*) parts of Alabama and Mississippi, and (*c*) isolated areas in North and South Dakota, Nebraska, and Colorado,

In composition these chalks, or "rotten limestones," vary from a rather pure calcium carbonate, low in both magnesia and clayey materials, to an impure clayey limestone requiring little additional clay to make it fit for use in Portland-cement manufacture. Analyses, quoted from various authors, of a number of these chalky limestones are given in the table below and will serve to show their range of composition.

Chalky limestones.

	Demopolis, Ala.	San Antonio, Tex.	Dallas, Tex.	White Cliffs, Ark.	Yankton, S. Dak.	Milton, N. Dak.
Silica	12.13	5.77	23.55	7.97	8.20	9.15
Alumina	4.17	2.14	1.50	1.09	7.07	4.80
Iron oxide	3.28					2.30
Lime carbonate	75.07	90.15	70.21	88.64	83.59	63.75
Magnesium carbonate	.92	.58	.58	.73	n. d.	1.25

Materials of this type are, in general, excavated and pulverized more cheaply and more easily than hard limestones. They also have an important advantage over the marls of the Northern and Central States in that they carry practically no water.

Portland-cement plants are now operating on materials of this type in the States of Alabama, Texas, Arkansas, South Dakota, and North Dakota, and it seems probable that these easily handled materials will soon prove to be the basis of an important industry.

Pure (hard) limestone and clay or shale.—Hard limestones are used, with clays or shales, in the manufacture of Portland cement. This combination of materials was first employed in New York State, but many plants now utilize it. The expense of excavating hard limestone is of course usually greater than that of dredging marl, but this disadvantage is counterbalanced by the fact that the limestone, as quarried, is usually dry, while marl may carry over 40 per cent of water. Analyses of typical materials of this group are given in the table below, the first four being of limestone, the others of clays or shales.

Limestone, clay or shale.

	Limestones.				Clay or shale.			
Silica	1.72	0.86	0.56	0.40	63.56	55.80	56.30	60.00
Alumina	1.63	.63	1.23	.44	27.32	30.20	29.86	23.36
Iron	6.59	1.03	.29					4.32
Lime carbonate	90.58	97.06	97.23	97.99	3.60	2.54	.....	1.70
Magnesium corbonate	.....	.....	.75	.42	2.60	.....	.....	1.50

Caustic soda waste and clay.—Precipitated lime carbonate, occurring as a by-product from alkali manufacture, has received considerable attention as a Portland-cement material. In combination with clay it is used at several plants in England and the United States.

Slag and limestone.—Blast-furnace slags running high in lime are valuable as one of the constituents of a Portland-cement mixture. The slag is a by-product and as such can be furnished cheaply to the cement plants. Usually the slag is granulated as it flows from the furnaces and mixed with a pure limestone. The mixture is then ground finely and burned in rotary kilns.

The slag can be furnished cheaply, but during granulation, which is effected by causing the molten slag to fall into troughs or tanks containing cold water, it takes up 20 to 40 per cent of water and this must be almost entirely driven off before grinding can be attempted. To partly counterbalance this disadvantage, the lime in the slag is present as oxide, not as carbonate, so that there will be a slight saving in the fuel required in the kilns.

EXCAVATION OF THE RAW MATERIALS.

The natural raw materials used at present are obtained by one of three methods—(*a*) quarrying, (*b*) mining, and (*c*) dredging. The choice between these different methods of excavation will depend partly on the physical character of the material to be excavated and partly on topographic and geologic conditions.

Quarrying.—The term "quarrying" is here used to cover all methods of obtaining dry material from open cuts or pits, whether the material excavated be a limestone, shale, or clay. Quarrying is the most natural and common method of obtaining cement materials. If marl (which is usually worked by dredging) be excluded from consideration, it is probably within bounds to say that 95 per cent of the materials used in American Portland-cement manufacture are obtained from open cuts or pits.

In the majority of quarries the material is blasted out and loaded by hand onto cars and carts. In a few limestone quarries this method is improved by using a steam shovel to do the loading. Steam shovels are more frequently employed for this purpose in shale quarries. In certain clay and shale pits, where the materials are of such character as to permit it, the steam shovel does all the work, both excavating and loading the materials, drilling and blasting being unnecessary.

The rock is usually shipped to the mill as quarried, without any treatment except sledging it to convenient size for loading. At a few quarries, however, a crushing plant is installed at the quarry, the rock being shipped to the mill as crushed stone.

Mining.—The term "mining" is used in distinction from "quarrying" to cover methods of obtaining materials by underground work through shafts or tunnels. Mining is rarely employed in dealing with materials of so little unit value as are employed in Portland-cement manufacture. Occasionally, however, when a thin bed of limestone or shale is being worked, its dip will carry it under such a thickness of other rock as to make mining cheaper than quarrying for that particular case. There are a few advantages in mining cement materials that serve to partly counterbalance the greater cost per ton as compared with quarrying. A mine can be worked economically in all kinds of weather, while a quarry is commonly in a more or less crippled condition for about three months of the year. Material mined is, moreover, always dry and clean.

Dredging.—The term "dredging" is here used to cover all methods of excavating soft and wet raw materials. Material of such a character as to be excavated by dredging will necessarily occur in depressions, and the deposit, in addition to containing much water diffused throughout its mass, may be covered by a more or less considerable depth of water. This may involve draining the deposit before dredging can be commenced.

The only cement material extensively worked by dredging in the United States is marl. Occasionally clay is obtained from deposits covered by more or less water, but this is rarely practiced unless marl occurs associated with the clay. In dredging marl the excavator is usually mounted on a barge, which floats in a channel resulting from

previous excavation. Occasionally the shovel is mounted on a car running on tracks laid along the edge of the marl deposit.

The material brought up by the bucket may be sent to the mill in two different ways. At plants using dome or chamber kilns, or at other plants where the cement mixture is not made very wet, the excavated marl is usually deposited in cars, hauled to the mill by horse or steam power. At plants using a really wet process, however, it is probable that the second method is more economical. This consists in dumping the marl into tanks on the bank, adding sufficient water to make it flow readily, and pumping the fluid to the mill.

Transportation to mill.—The method of transporting the raw materials to the mill is almost entirely a question of distance and grade. Most cement mills are located close to at least one of their raw materials, and the commonest method of handling and transporting is probably to load the material on cars by hand at the quarry face, and run the cars by horse power to a point where they are fastened to a cable, which hauls them up to the mill. Where a hill or ravine intervenes between quarry and mill, or where there is a great difference in elevation between the two points, it is probable that an aerial cableway will prove to be the most satisfactory method of transportation.

Cost of raw materials at mill.—The most natural way, perhaps, to express the cost of the raw materials delivered at the mill would be to state it as being so many cents per ton or cubic yard of raw material, and this is the method followed by quarrymen or miners in general. To the cement manufacturer, however, such an estimate is not so suitable as one based on the cost of raw materials per ton or barrel of finished cement.

In the case of hard and comparatively dry limestones or shales it may be considered that the raw material loses 33⅓ per cent in weight on burning. Converting this relation into pounds of raw material and of clinker we find that 600 pounds of dry raw material will make about 400 pounds of clinker. Allowing something for other losses in the process of manufacture, it is convenient and sufficiently accurate to estimate that 600 pounds of dry raw material will give one barrel of finished cement. These estimates must be increased if the raw materials carry any appreciable amount of water. Clays will frequently contain 15 per cent or more of water, while soft chalky limestones, if quarried during wet weather, may carry as high as 20 per cent. A Portland-cement mixture composed of a pure chalky limestone and a clay might, therefore, average 10 to 20 per cent of water, and consequently about 700 pounds of such a mixture would be required to make one barrel of finished cement.

With marls the loss on drying and burning is much greater. Russell states[a] that according to determinations made by E. D. Campbell

[a] Twenty-second Ann. Rept. U. S. Geol. Survey, pt. 3, 1901, p. 657.

1 cubic foot of marl as it usually occurs in the natural deposits contains about 47½ pounds of lime carbonate and 48 pounds of water. In making cement from a mixture of marl and clay, therefore, it would be necessary to figure on excavating and transporting over 1,000 pounds of raw material for every barrel of finished cement.

From the preceding notes it will be understood that the cost of raw materials at the mill per barrel of cement will vary not only with the cost of excavation, but with the kind of materials in use.

In dealing with hard, dry materials extracted from open quarries near the mills the cost of raw materials may vary between 8 cents and 15 cents per barrel of cement. The lower figure named is probably about the lowest attainable with good management and under favorable natural conditions. The higher figure is probably a maximum for fairly careful management of a difficult quarry under eastern labor conditions. When it is necessary to mine the materials the cost will be somewhat increased. Cement rock has been mined at a cost equivalent to 10 cents per barrel of cement, but this figure is attained under particularly favorable conditions. The cost of mining and transportation may reach from this figure up to 20 cents per barrel.

With regard to wet marls and clays it is difficult to give even an approximate estimate. It seems probable, however, when the dead weight handled is allowed for, that these soft materials will cost almost as much, delivered at the mill, per barrel of finished cement, as the hard, dry limestones and shales.

PREPARATION OF DRY MATERIALS FOR THE KILN.

Whatever the raw materials employed, the object of all the operations that may be carried on prior to burning is simply to secure an intimate mixture of these materials and to insure that this mixture is finely ground and of proper chemical composition.

The character of the materials employed, however, has determined two general methods of preparing the mixture for the kiln, the differences between these methods being due almost entirely to the physical characteristics of the raw materials used. For this reason a general subject of preparing the materials for the kiln will be discussed under three heads:

(a) Mixing and grinding dry materials.
(b) Mixing and grinding wet materials.
(c) Mixing and grinding slag Portlands.

Drying.—Raw materials as they come from the mine or quarry will almost invariably carry appreciable, though often very small, percentages of water. In the case of hard limestones the water will usually range from one-half per cent to 3 per cent, rarely reaching or exceeding the higher figure. With the chalky limestones, which absorb water rapidly, the percentage can be kept down to 5 per cent or less,

reservoir sites lie at considerable heights above the river the large feeder canals would have great length. It has been suggested that the complete utilization of North Platte waters can be most successfully accomplished by diverting the entire flow at all times into a canal which will lie at a greater elevation than the reservoir sites which may be determined upon as available, and will serve as a feeder to all of the reservoirs. As a part of this scheme it is contemplated that all present or future canals shall derive their supply from this main canal or from the reservoirs which are fed by it. A necessary and economical modification of the scheme is to the effect that a portion, at least, of the waters of maximum floods should be diverted from the river into the minor canals, as it would be extravagant to construct the main canal of size to carry the entire volume of these maximum floods. A canal constructed of such size would be utilized to its full capacity only in the most exceptional years. It is doubtful if it would be wise to approach this size of main canal so nearly that the excess of maximum floods would all be taken from the river by the minor canals. In other words, until water has a value for irrigation considerably in excess of what may be expected in this generation in the North Platte Valley it will not be advisable to undertake to make available the entire discharge of the stream and to prevent all of the waste which ensues from the passage of the water down the river.

The considerable reservoir sites in Wyoming in which water may be stored for Nebraska lands lie on the north side of the river, so far as they have come to the attention of the writer. When we come to consider the high lands—that is, the lands higher than first and second bottom—it is found that a vastly larger acreage is susceptible of irrigation on the south side of the river than on the north. The advantages and disadvantages of each site as they shall be brought to light by surveys and other investigations should be carefully considered, in order to decide on which side of the river the large development shall be undertaken or whether the large potential supply of water shall be divided between the two sides.

OTHER RIVERS.

On the upper part of Republican River, and on its tributary, Frenchman River, agriculture without irrigation is an occupation yielding uncertain results. At the same time agriculture with irrigation meets with satisfactory returns. Hence the irrigation ditches which have been constructed are used regularly to the extent of the supply of water available from the streams, and better and better results are obtained by the irrigators from year to year. The problem of storing and utilizing the supply, which under present conditions is not available, is now an important one in the minds of the users of water, and

it is safe to assume that some attention will be given to it by the engineers of the reclamation service in the near future. The conditions are such that little can be done toward the solution of the problem until it has been taken up on the ground in detail. Since no part of the drainage area is mountainous, the floods are due entirely to rainfall, which is mainly of irregular occurrence in point of time and of uncertain magnitude.

The unseasonable discharge which at present runs to waste would be the principal and most definite item to be considered. It may be estimated roughly that if this could be conserved the irrigating capacity of the streams would be doubled. This would be possible only in case sites suitable for the storage of 75,000 to 100,000 acre-feet can be found. No surveys or examinations other than the most casual have been made to determine if such sites can be located.

Reckoned on the basis of total discharge, Loup River is second to the North Platte. The seasonal distribution of its discharge is, however, markedly different. The discharge arises mainly from the precipitation in the sand-hill region, which is of such a character that even the heaviest rains sink at once and almost entirely into the soil and the water reaches the streams in the form of springs, resulting in a flow which is subject to only relatively small variations. The total discharge of the Loup at the gaging station a few miles above its mouth, during the irrigation season of the year, is about three-fourths that of the North Platte for the same period. Even in the case of a discharge presenting such a marked degree of uniformity it can not of course be assumed that the demand and the supply will be exactly conformable to each other. If, therefore, irrigation from the stream is ever developed to such a point that the supply is at times unequal to the demand upon it, the question of storage will have to be considered. There is a prospect that practically the entire flow of the river will be used at an early date for the development of power at some point in its lower course or after the water has entered the bed of Platte River. If projects now in contemplation are carried to completion and their rights to the use of the water are perfected, no storage except that of the surplus waters of occasional floods can be accomplished without considering these rights. This would complicate the problem of the storage of the winter flow. It is not known that any consideration has been given to the problem of extensive water conservation on this stream. The character of the stream in respect to the extent and uniformity of discharge and the relatively small amount of irrigation development which has taken place on the stream and its tributaries have combined to make the storage of its waters seem less important than that of the waters of some of the other streams. It may be expected that when an investigation along this line is taken up on

Composition of mixture.—The cement mixture ready for burning will commonly contain from 74 to 77½ per cent of lime carbonate or an equivalent proportion of lime oxide. Several analyses of actual cement mixtures are given in the table below. Analysis No. 1, with its relatively high percentage of magnesia, is fairly typical of Lehigh Valley practice. Analyses Nos. 2 and 3 show mixtures low in lime, while analysis No. 4 is probably the best proportioned of the four, especially in regard to the ratio between silica and alumina plus iron. This ratio for ordinary purposes should be about 2½ to 1, as the cement becomes quicker setting and lower in ultimate strength as the percentage of alumina increases. If the alumina percentage be carried too high, moreover, the mixture will give a fusible, sticky clinker when burned, causing trouble in the kilns.

Analyses of cement mixtures.

	1.	2.	3.	4.
Silica	12.62	13.46	13.85	14.77
Alumina and iron oxide	6.00	(?)	7.20	4.35
Carbonate of lime	75.46	73.66	73.93	76.84
Magnesia oxide	2.65	(?)	(?)	1.74

Burning the cement mixture.—After the cement mixture has been carefully prepared, as described on preceding pages, it must be burned with equal care. For a time various types of kiln were in use, but now practice in burning at the different cement plants is rapidly approaching uniformity.

The kiln in which the material is burned is now invariably of the rotary type, the rotary process, which is essentially American in its inception and development, being based on the substitution of machine for hand labor wherever possible.

As at present used, the rotary kiln is a steel cylinder, commonly 60 feet in length and 6 feet in diameter. This is set in a slightly inclined position, the inclination averaging about one-half inch to the foot. The kiln is lined, except near its upper end, with very resistant fire brick to withstand the high temperature to which its inner surface is subjected.

Fuel, either powdered coal, oil, or gas, is injected at the lower end of the kiln, while the cement mixture is fed in at the upper end. The kiln is slowly revolved about its axis, and this revolution, in connection with the inclination at which the kiln is set, gradually carries the cement mixture to the lower end of the kiln. In the course of this journey the intense heat from the fuel first drives off the water and carbon dioxide from the mixture and then causes the lime, silica, alumina, and iron to combine chemically to form the partially fused mass known as "cement clinker." This clinker drops out of the lower end of the kiln, is cooled so as to prevent injury to the grinding machinery, and is then sent to the grinding mills.

In dealing with a dry mixture one rotary kiln will produce 150 to 200 barrels of cement per day of twenty-four hours, with a fuel expenditure of 105 to 140 pounds of powdered coal per barrel. If the cement mixture carries much water, as is usually the case where marl is used as one ingredient, the output of the kiln is greatly reduced and its fuel consumption very largely increased.

Grinding the clinker.—After cooling the clinker, which is usually effected by exposing it to the action of cool-air currents, it is sent to the grinding mills. In order to retard the setting time of the cement sufficiently to give good results in use, about 2½ per cent of gypsum is commonly added before the grinding of the clinker is commenced. Clinker grinding is commonly carried on in two stages, the first reduction taking place in Griffin mills, ball mills, or komminuters. These reduce the clinker to a size to pass a screen with 20 or 30 meshes to the linear inch. The material passing these screens is then finely pulverized in tube mills, which reduce it fine enough for about 95 per cent to pass through a screen with 100 meshes to the linear inch.

The ground cement is sent to storage bins, from which it is drawn out as needed and packed in sacks, bags, or barrels.

Present sources of cement supply.—At present the following cement plants are in operation in the West:

(1) Standard Portland Cement Company plant is located at Napa Junction, Cal. Ten rotary kilns are in operation, giving a nominal capacity of 2,000 barrels per day. The materials used are an argillaceous limestone and a pure limestone. Brand, "Standard." J. F. Bachmann, manager; L. T. Bachmann, chemist.

(2) Pacific Portland Cement Company plant is located at Cement, a few miles east of Suisun, Cal. Eight rotary kilns are in operation, giving a nominal capacity of 1,600 barrels per day. The materials used are a pure limestone and clay. Brand, "Golden Gate." Morris Kind, manager; C. J. Wheeler, chemist.

(3) California Portland Cement Company plant is located at Colton, Cal. Three rotary kilns are in operation, giving a nominal capacity of 600 barrels per day. Materials used are pure limestone and clay. Brand, "Colton." A. Neu, chemist.

(4) Portland Cement Company of Utah plant is located at Salt Lake City, Utah. Four rotary kilns are in operation, giving a nominal capacity of 800 barrels per day. The materials used are an argillaceous limestone and a pure limestone. Brand, "Red Diamond." L. M. Barley, manager; C. S. Batchelder, chemist.

(5) Portland Cement Company of Colorado plant is located at Portland, a few miles east of Florence, Colo. Six rotary kilns are in use, giving a nominal capacity of 1,200 barrels per day. The materials used are an argillaceous limestone and a pure limestone. Brand, "Ideal." A. Moore, manager; F. W. Brown, chemist.

(6) Iola Portland Cement Company, plant is located at Iola, Kans.

Twenty-one rotary kilns are in use, giving a nominal capacity of 4,200 barrels per day. Materials used are limestone and shale. Brand "Iola." O. Gerlach, chemist.

(7) Kansas Portland Cement Company, plant is located near Iola, Kans. Eight rotary kilns are in use, giving a nominal capacity of 1,600 barrels per day. Materials used are limestone and shale. E. C. Champion, superintendent and chemist.

THE LEGAL STATUS OF IRRIGATION.

By H. L. Holgate.

IRRIGATION AND LITIGATION.

In 1899, according to the reports of the Twelfth Census, there were in the States and Territories usually reckoned arid 102,819 irrigated farms. This means that, at a low estimate, over half a million people in those States and Territories were then directly interested in irrigation. There were more than 7,000,000 acres irrigated, and the value of the crops produced thereon exceeded $84,000,000. The irrigation systems in operation represented a construction cost of more than $64,000,000.

These are large figures, and they do not include the statistics for the semiarid and humid regions, the increase since 1899, or the number of people and amount of investments indirectly dependent upon irrigation. They make a creditable showing for the courage and enterprise of the western farmer, but when we learn upon what a frail and uncertain legal foundation the great and immensely important structure of irrigation stands, we are unable to pay a like tribute to the statesmanship of the western lawmaker. Happily there is at present a tendency to correct the statutes, and within recent years a number of States have made fairly successful attempts to perfect laws governing public waters.

The history of water diversion in the West is so replete with appeals to the courts that one unconsciously associates irrigation with litigation. The money spent in litigation has been enormous. In taking the Twelfth Census an attempt was made to ascertain the cost of litigation over water rights in 1899. The work was not completed, as it was found impossible, in a great many cases that were continued through a number of years, to segregate the expenses of 1899. In California the owners of 73 irrigation ditches reported an expense during that year of $125,000 for court and lawyer fees incurred in defending water rights. Presumably the defendants spent as much more. Here we have, without taking into account the cases unreported, over a quarter of a million dollars spent in one State in a single year in litigation on this subject, and few of the decisions, when made, were conclusive or gave the victors a sense of security.

There is a woeful lack of uniformity in the legislation had in the several States and Territories, and there are imperfect laws in individual States. The need for reformation is imperative, the more so since the National Government has undertaken the heavy responsibility of constructing and operating reclamation works.

ACQUISITION OF WATER RIGHTS.

The necessities of the West have given rise to a new legal doctrine relative to water diversion. The requirements, originally of mining and later of agriculture, could not be satisfied by the operation of the common-law doctrine of riparian rights which exist simply by virtue of ownership of lands adjacent to a stream.

The new doctrine of appropriation, under which water rights are acquired by actual diversion for beneficial use, independent of riparian ownership, is now recognized where irrigation is generally practiced. In Arizona, Colorado, Idaho, New Mexico, Nevada, Wyoming, and Utah the law of riparian rights has been repudiated, and in Nebraska it is limited to the humid portion of the State. In California, Kansas, Montana, the Dakotas, Oregon, and Washington both bases of water rights are recognized either by statutes or court decisions, and there is a resultant legal confusion that a mass of judicial decisions has not untangled. It is impossible to define the relationship between the operations of the two doctrines in these States. In Washington, which has a somewhat comprehensive statute relating to the appropriation of water rights, it has been held that long and unprotested usage by an appropriator is no bar to ouster by a riparian owner, while in Oregon, a neighboring State, where the written law recognizing the doctrine is brief and limits the right of appropriation to corporations, the courts have recently declared that riparian owners can not interfere with appropriators of water for irrigation who have used it without protest.

STATE CONTROL OF WATERS.

In Wyoming and western Nebraska, and, by an act of the last legislature, in Utah, the State is absolute administrator of streams and the diversion of water therefrom. The appropriator must, upon written application, secure a permit from the State engineer before beginning construction of diversion works, and upon satisfactory completion of the works he receives a certificate which is a title deed to his water right.

In Colorado and in Nevada, under its very recent statute, the State administration of streams does not begin until the appropriation is actually made, and the irrigation officials do not then interfere with the use of water except upon a written complaint, but the authorized State officials are clothed with sufficient power to prevent wasteful use

and insure equitable distribution to appropriators according to their decreed rights. The water commissioners of Washington have somewhat similar authority.

ADJUDICATION OF PRIORITIES.

Nebraska, Nevada, Utah, and Wyoming have enacted laws governing the adjudication of water rights based upon very similar principles. Wyoming initiated this system, and the laws of Nebraska, Nevada, and Utah are copies of, or are based upon, the Wyoming statute.

The Wyoming law provides for the adjudication of all claims by the board of control. Preliminary to the determining of priorities upon a stream the State engineer makes thorough surveys and measurements, and prepares maps and tables showing the location and size of the ditches, the area irrigated, and the measured flow of the streams. Claimants are notified to file claims and proofs, and the board establishes the priority and extent of all the water rights on the stream and its tributaries. Appeals from the decision of the board of control may be made to the courts, but in Wyoming and Nebraska (where a similar law has been in operation for a number of years) very few appeals have been made to the courts.

Nevada's new irrigation law is different from that of Wyoming in respect to the adjudication of water rights only as to the determining authority, which is vested in Nevada in the State engineer.

Under the Utah law adopted this year, the district court of each county is made the arbiter of the various claims on streams located wholly within the county, and where the stream traverses two or more counties the district court of the county most convenient to all the appropriators from the stream adjudicates priorities. Other than this the method agrees practically with that of Wyoming. The tables, maps, and drawings of the State engineer are to be considered prima facie evidence of the facts.

The method adopted by Colorado in 1879 and amended by subsequent legislation makes the adjudication the duty of the several district courts, and decisions are based upon the evidence offered by the parties in interest.

WATER APPURTENANT TO LAND.

Under the common-law doctrine, water is appurtenant to the land. The riparian owner can, it is true, assign his right, but the assignment is operative only against himself and his successors and gives the assignee no rights as against lower riparian owners.

In the West generally a water right is regarded as a species of realty and can be transferred by a deed. As a rule a deed of land

is presumed to include the water right, unless the latter is specifically excepted. The more recent Colorado decisions hold that the inclusion or noninclusion of water rights in the conveyance of land where the water rights are not mentioned in the deed is a matter of intent.

In Wyoming, water rights are treated as part and parcel of the land and inseparable therefrom. The board of control refuses to recognize transfers of water rights apart from the transfer of land. This attitude has been attacked in the courts, and the lower court has held that the right to the use of water is a property right and as such is transferable. The question was appealed to the supreme court, but has not yet been decided.

Some of the States, in their laws providing for the operation of the Carey Act, make the water appurtenant to the land reclaimed under that act.

In an Idaho court it was held a few months ago that a user of water under a contract for rental is not an appropriator under the meaning of the laws of the State; that the dedication to use applies to the land only upon which the use is had, and that the right is not transferable.

UNDERGROUND WATERS.

In the light of judicial decisions it can be accepted as a rule of law that the waters from underground streams may be appropriated and priorities in rights to waters therefrom established. The rule is recognized by statute in Kansas, and means for protecting such water rights are provided.

In trying cases involving rights to water developed from underground sources, the courts have to decide the difficult question of fact whether the water is from a subterranean stream having a well-defined and ascertainable channel, or from a stream of unknown channel, or is percolating water. Quoting from a decision of the supreme court of Utah, 1900: "The rule of law applying to the appropriation of surface waters does not apply to percolating waters which, being a part of the earth, have no characteristics of ownership distinct from the land itself." The water from a stream whose channel is unknown is treated, perforce, in law as percolating water.

In practical contradiction to the Utah decision just quoted, is a New York decision made the same year, which states in effect that one who on his own land pumps and disposes of water in such quantities as to materially lower the underground waters of neighboring lands, to the injury of crops grown thereon, may be enjoined as for trespass.

IRRIGATION ENGINEERS.

The States having State engineers who have administrative power over streams are Colorado, Nebraska, Nevada, Utah, and Wyoming. In Nebraska the State engineer is not so named, as the number of State

officers is limited by the constitution, but there is a board which has control of the streams and of their use in irrigation, and the secretary of the board is engineer in fact.

In these States the State is divided into water divisions, and these into water districts, under the administration of water commissioners. The State engineer has supervision over the division superintendents and the water commissioners, and is the court of appeals from their administrative decisions. Besides supervising the administrative duties of these irrigation officers the State engineer measures streams, collects irrigation data, prepares maps, and makes necessary reports. In Wyoming, Nebraska, and Utah he grants permits to intending appropriators, and in Utah he issues the water-right certificates upon completion and acceptance of the diversion works. In Wyoming and Nebraska the board of control issues the certificate.

Idaho has a State engineer, but he has no administrative control of the streams, and his work is chiefly statistical. In South Dakota the office of engineer has been superseded by that of irrigation instructor, whose duties are experimental and statistical. He must be appointed by the governor from the experimental station staff or Agricultural College faculty.

The new Nevada law introduced a novelty in State legislation. It provides that the governor's appointee to the office of State engineer must be nominated by the Secretary of the Interior or the Director of the Geological Survey, if those officials choose to exercise the privilege.

The other irrigation States have no State engineers.

RECLAMATION SURVEYS.

[Report of the committee on reconnaissance and surveys, consisting of Jeremiah Ahern, Cyrus C. Babb, and J. H. Quinton.]

The engineer detailed to examine an irrigation project is given considerable discretionary powers as to lines of investigation. However, in order to have methods of surveys in different sections fairly comparable, the following instructions have been formulated, though they will necessarily be modified occasionally to suit the varying conditions of different projects.

ENGINEER'S EXAMINATION.

The engineer should first consider the amount and quality of the land and satisfy himself that there is available sufficient water in addition to that already appropriated to be utilized, either as perennial flow or by means of storage. The amount of water available can be determined by a few rough measurements of the main drainage channel, and by a careful study of the rainfall and run-off records and of all maps

showing the drainage areas tributary to the district to be irrigated. In some cases the same investigations should be made in adjacent drainage areas which might be made to contribute part of the water.

If the project contemplates storage the engineer should consult all records of rainfall and run-off in the drainage basin. If there are no such records he should form some idea of the quantity of water which passes a given point on the main drainage line by examining on the ground all high-water marks and getting from the people who have lived in that neighborhood all the information possible as to the duration of floods. This will enable him to decide whether it is advisable to establish the necessary rain gages and gaging stations for determining the run-off for a few years.

The nature of the drainage area, whether forested or otherwise, should be taken into account. In order to ascertain its general physical features an examination requiring several days' time may be necessary. In this way a trustworthy general idea of the possibilities of the drainage area may be formed. In the case of a large drainage basin where there may be a choice of reservoir systems or canals a map compiled from railroad and public-land surveys and a barometric reconnaissance may be necessary.

On this trip all possible reservoir sites, whether on the main stream or elsewhere, should be examined in a rough way by hand level and aneroid, and the nature and approximate dimensions of dam and reservoir sites, as well as quality, quantity, and availability of material for construction of dam should be noted.

If all these conditions are found sufficiently favorable to justify continuing the examination of the project, the next thing to be determined, in a rough way, is the probable cost of conveying the water to the land. This can be done only by a careful reconnaissance along the approximate line of the canal. Experience on construction in work of this class is of great value in forming a judgment in this matter.

These points having been determined a careful examination of the land may be made, and all its advantages and disadvantages noted. The principal points to be considered are the quantity of land which can be irrigated, the ownership of the land (whether largely private or not), the amount of water used from the stream upon which the project must depend, the fertility of the soil, the difficulties of preparing the land for irrigation, the crops most suitable for the soil and the climate, the number of crops that can be counted on annually, the probable value of said crops, and whether the cost per acre for conveying the water to the land will enable the farmer to keep himself and family and pay back the cost of the works to the Government in ten years.

WITHDRAWAL OF LAND.

If the project be deemed worthy of further investigation recommendations should be made for the withdrawal of the land under the reclamation law of June 17, 1902 (32 Stat. L., p. 388), as described in section 49 of the survey's book of Instructions, page 96.

Special attention should be paid to the method of describing the lands by legal subdivisions, as indicated in paragraph 3 of section 49 of the Instructions. There is danger of error in transcribing descriptions if they are written out consecutively in solid lines. The tabulation indicated in the Instructions is that used by the General Land Office, and has been found the most satisfactory after many years of experience.

In making requests for withdrawals it is not necessary to omit tracts which have been disposed of by the Government, as the claims to such tracts would not be affected by a withdrawal of an entire section or of an entire township including them. The withdrawal is made in such form as to exclude any tract the title to which has passed from the United States, and also to provide for the withdrawal going into effect if any claim is afterwards canceled or abandoned.

The circular issued July 21, 1903, contains instructions for retracing the lines of the public-land surveys. Attention was called therein to the importance of not interfering with vested rights where the lands have been settled upon. In such cases, therefore, where corners can not be found, there will be no authority for reestablishing them, because such action would, at the same time, affect the rights acquired by a settler or patentee of other lands bounded by the same line. Under these circumstances the engineer should run out the line as if the entire tract were public land, so that his plane-table sheet may show the location, but no corners are to be established. In case public land adjoins lands which have been granted to railroads, and which have not been disposed of to private parties, the lines of the public surveys may be rerun and the corners reestablished if necessary. The fact that the adjoining tract belongs to the railroad need not be regarded as an obstacle in such a case.

The plane-table sheets should show the public-survey lines in all cases of surveys for irrigation works or irrigable lands. The corners should be marked so as to distinguish those found on the ground, those not found, and those reestablished.

FLYING-LINE RECONNAISSANCE.

If the preliminary examination shows that the water supply is sufficient and that the relative positions of land and water are such that the project seems feasible, the next step is to make a flying-line reconnaissance. The object of this reconnaissance is to determine whether

or not a preliminary survey of the project should be made and the character and extent of such a survey—that is, whether it should consist of one or more preliminary transit lines through certain portions of the tract or a topographic survey of the entire area. In other words, it will be necessary to determine approximately the relative elevations of the controlling points and the area of the land that may be reclaimed. The reconnaissance should start from a controlling point, which may be the proposed point of diversion, the outlet from the reservoir, the highest point of the irrigable land, or the lowest pass of a ridge through which the canal must go.

The instruments best adapted for this work are a light mountain transit, provided with a vertical arc and level attached to its telescope, and a stadia rod. The survey consists simply of running through the area one or more traverse lines so located that they will give the necessary information. Distances between stations of these lines should be obtained by stadia, directions by the magnetic needle, and differences of elevation by vertical angles. Summits of hills, lone trees, houses, or other prominent objects, whose position may be of value in locating topographic features, should be "cut in" and vertical angles to them measured. If the area has been covered by a land-survey, connection with its corners should be made whenever practicable.

In work of this kind three men are required, a transitman, a rodman, and a teamster. If the character of the topography permits, the transitman should travel between stations in a light wagon or on horseback, and the rodman should be provided with a saddle horse. In the above-described manner a reconnaissance can be made rapidly and cheaply which, if the lines and points located are well selected, will furnish at a minimum expense sufficient data for the determination of the question in hand. By combining the information thus obtained with that of the public-land surveys a good reconnaissance map of the area may be made.

RESERVOIR AND DAM SITE SURVEYS.

Surveys of reservoir and dam sites should preferably be made by plane-table methods, except where the site is densely wooded, in which case transit and stadia methods should be used. In cases of reservoirs, the standard scale generally to be adopted is 1,000 feet to an inch, with 10-foot contours. In the case of dam sites the standard scale is 20 or 50 feet to an inch with 5-foot contours, except where the detail of topography is not great, when a smaller scale may be used.

All reservoir surveys should be made with sufficient accuracy to determine the approximate capacity of the reservoir and contents of of the dam with a possible error of not to exceed 10 per cent.

One or more permanent bench marks should be established near the

proposed dam site; the elevation should be determined with reference to the river bed or water level, and a line of levels should connect the bench mark with the top contour of the survey. Wherever practicable this bench mark should be connected by a checked line of levels with a permanent bench mark of the topographic branch of the survey, in which case all elevations will be referred to mean sea level. A line of levels should also be run up the valley of the reservoir, stakes being driven and flags erected at every point where a 10-foot contour crosses the line. If the reservoir be wide or timbered, so that the vertical angle connections with these contour crossings can not be made frequently and conveniently, additional points near the top contour should also be established with the Wye level.

A base line of such length and so located that a well-conditioned, graphic triangulation may be expanded from it should be measured in duplicate by a steel tape, and from this base line a careful plane-table triangulation should be executed. Before beginning the triangulation all stations should be marked by artificial signals. Where a reservoir site, on the scale adopted, covers more than three consecutive plane-table sheets, all the angles of a triangulation system should be measured by a transit, and the relative position of the stations should be computed and plotted on the plane-table sheets before beginning the plane-table surveys.

Secondary points for the control of the sketching can be located by stadia methods. Hills, buildings, lone trees, or other recognizable objects should be utilized as far as available, and be located by intersection. Secondary points should be skillfully distributed so as to establish controlling points in the topography, such as change of slope, important junction or bends in drainage lines, hills, passes, etc. A careful search should be made for corners of the public-land surveys, and all found located accurately on the plane-table sheets.

Where a reservoir site is so densely wooded as to absolutely prohibit any field sketching and necessitate the cutting of lines for all sights and the locating of a very large number of points to establish the contours, transit and stadia methods should be used. The method generally used is to run a traverse line up the valley of the reservoir site, and one along the top contour of the survey; and where the site is not too large points on these lines may be connected by traverse lines, differences of elevation being obtained by the hand level, and distances determined by pacing. Where the reservoir is large it will be necessary to run these traverse lines by transit and stadia.

PRELIMINARY CANAL SURVEYS.

Before beginning a preliminary canal survey the chief of party should become thoroughly familiar with the country through which

the canal must pass. Party organization for a preliminary survey will usually consist of one chief of party, one transitman and levelman, one plane-table man, and the necessary rodmen and camp assistants.

The chief of party will give the general location of the line. The instrument man, whose party will comprise, in addition to himself, one rodman, one stakeman, and axmen if necessary, should place stakes with a level along the line of the canal, estimating approximately the grade. These stakes should be placed from 200 to 300 feet apart and be marked with a consecutive number and elevation. Distances between the stakes should not be measured in this first step, but should be estimated. This line should not follow the grade contour on the ground, but should be located as nearly as possible where the final line would cross ridges or ravines. The second day's work of this party should consist in locating the stakes with transit and stadia, running an azimuth line in preference to deflection angles. The transit notes should be platted in camp from day to day as the work progresses, on a long roll of detail drawing paper, generally on a scale of 400 feet to the inch. These platted points should then be transferred to plane-table sheets by placing the said sheets under the drawing paper and pricking through with a needle point. Points thus transferred should be marked with the consecutive stake number and the elevation.

These sheets should then be taken into the field by the plane-table man, whose party will include one rodman, and one station assistant to keep notes and work up the elevations and horizontal distances from the vertical angles and stadia distances. The topography in 5-foot contours should be mapped with alidade and stadia for a distance of 300 to 500 feet on both sides of the line. The sheets should overlap an inch or two at the most convenient angle, at or near the left side of the lower sheet, so that the line and the topography may be continuous when the sheets are superimposed in their proper order.

Wherever the line crosses a watercourse, however small, the plane-table man should make notes—preferably on the plane-table sheets—of the approximate area of the drainage basin above the point of crossing, and estimate from the position of drift, etc., the probable maximum discharge of the stream at the point of crossing. This information is of great value in the office when making designs for crossing watercourses. Wagon roads, railroads, and other construction intersecting the line should be noted.

The level work should be of the degree of accuracy required by the regulations of the topographic branch; that is, the limit of error in feet should not exceed $0.05\sqrt{\text{distance in miles}}$. Permanent bench-mark posts should be established on the line of survey at intervals

not greater than 3 or 4 miles and, wherever practicable, be connected with the bench marks of the topographic branch. Temporary bench marks should be established not more than a mile apart in open country and at every 1,000 feet in rough or broken country. Connections should be made as often as possible with the located corners of the public-land surveys, and with the primary triangulation of the topographic branch where it exists. The three-point location is generally the most convenient method for making the later connection.

Another method of survey, which may be found advantageous under certain conditions, is described below:

The first party should make the location of the canal line with a Wye level, setting stakes from 500 to 1,000 feet, suitably marked with the ground elevation and with a flag, so that they can be easily seen at some distance. The line should follow the approximate grade of the canal and not run as a simple contour line.

The second party should locate this line with plane table, distances being obtained by stadia and orientation or direction with the magnetic needle. Frequent connections should be made with section corners, where the area has been covered by the public-land surveys. The scale of map may be 2,000 feet to the inch. The contour interval should be 10 feet, from which the slope of the ground can be obtained and estimates of quantities made. By this method from 4 to 6 miles a day can be run, thus easily keeping up with the level party.

The location of the line will be known from time to time as the work progresses, and it will not be necessary to afterwards plot up the line, as is the case when transit methods are adopted. Connections with the corners of the public-land surveys can also be more quickly made, as the land lines can be projected ahead of the work after the first one or two corners have been found.

Such a party should consist of one levelman and his rodman, one plane-table man with his rodman, one teamster, and one cook.

Observations on a circumpolar star or on the sun should occasionally be taken as the survey progresses, in order to check the azimuth of the line. These observations, if possible, should be made at a station on the line.

Where only a hasty reconnoissance can be made of the canal line, the topography, instead of being mapped with a plane table, may be sketched in a notebook, only one man with hand level being required for that purpose. He will generally keep up with the transit party.

Along canyon sections the scale of 400 feet for the plane-table work is hardly large enough, and if an accurate estimate is desired a larger scale of 200 or 100 feet to the inch will be necessary. Experience has shown that through open country, with the recommended scale of 400 feet, the plane-table party can generally keep up with the locating party if the levels are run one day and the same line located with transit the following day.

TOPOGRAPHIC SURVEYS.

Wherever the topography is such that preliminary canal lines can not be easily located by the method given in sections 27 and 28 of the Survey's book of Instructions, as in a very difficult rolling country, a topographic survey of the entire area should be made, and the preliminary location of canal lines made on the resulting topographic map. For such a survey plane-table methods should always be used, and the general methods of topographic surveying given on pages 56–81 of the Instructions should be employed in so far as they are applicable to the work.

The topographic branch of the Survey has extended primary triangulation and primary leveling over various parts of the country. Before a topographic survey of a project is begun information should be obtained from the Washington office as to whether these primary surveys have been extended over the area, and if so, such of their data obtained as will expedite the work.

SURVEYING IRRIGABLE LAND.

The same general methods prescribed for topographic field work (pp. 56–81 of Instructions) should be used in surveying irrigable land under a proposed canal line. Where no primary triangulation exists it will be necessary to measure a base line and expand the plane-table triangulation from it. As in the case of reservoir surveys, where the area covers more than three consecutive plane-table sheets, an instrumental triangulation should be made with a transit or theodolite.

The scale of field sheets should not be less than one-half mile to an inch, and the contour interval not greater than 10 feet. As in reservoir surveys, connection should be made with all land-office corners.

TRIANGULATION.

Where it is necessary to measure the angles of a system of triangulation for the control of the topography of a large area of irrigable land, and where the instrument used reads to 20 or 30 seconds, the following method is generally of sufficient accuracy:

The signals are supposed be in the order of A B C—A.

First set ....
- Telescope direct—read on A B C—A.
- Telescope reversed—read on A—C B A.

Second set ..
- Shift the limb 90°.
- Telescope reversed—read on A B C—A.
- Telescope direct—read on A—C B A.

If more than two sets are necessary the limb should be shifted, between sets, 180° divided by the number of sets.

In all cases read both verniers.

Each set is complete in itself.

Observations on a circumpolar star for azimuth should be made at the beginning and end of the triangulation.

FINAL LOCATION.

In finally locating a waterway many things must be considered which are necessarily matters of judgment on the part of the engineer.

The first and most important is the velocity which the water must have for a given material. This involves changes of gradient and changes of cross section for different classes of material.

If the velocity is too high the material of the bottom and sides will be eroded and carried to places whence it will have to be removed at some expense every year. If the velocity is too low, there will be an unnecessary outlay in first cost of excavation and of maintenance in removing deposits of sediment. Good judgment is essential in this matter, and it is probably better to err on the side of safety by fixing a low velocity instead of a high one. This decision may be modified, if there is much sediment in the water, as a low velocity will allow the sediment to accumulate where it may affect the capacity of the canal, and entail an annual expense for cleaning which would be unnecessary if the velocity were higher.

The following limiting values for mean velocity in canals are given, although they may be changed if in the judgment of the engineer such change is warranted:

2 to 3 feet per second for ordinary soils; 1.75 feet per second, minimum, unless water is very clear, when a lower velocity may be used.
3 to 3.5 feet per second in ordinary firm soils.
5 to 7 feet per second, safe in hardpan and soft rock.
8 to 10 feet per second in cement work.
15 feet per second in solid rock.

The velocity and the quantity of water to be carried being fixed, the area of cross section is known. This area may, however, be put into many different shapes, which must vary according to the material encountered. In the simplest case, the area being a rectangle, there are but two factors to consider—the breadth and depth. In trapezoidal channels an additional factor has to be considered—the slope of the sides. In the most general case, if

A equals the area,
B equals the base,
d equals the depth,
m equals the ratio of slopes, $\frac{\text{horizontal}}{\text{vertical}}$
The area A equals Bd plus md^2.

There are here three variable quantities, and it becomes necessary to fix two of them before the third can be found.

It is evident that m is fixed between narrow limits on account of the material, and that B and d may vary considerably. As the width of the base is of minor importance compared with the depth of the water, it is best to fix the latter as soon as the velocity is decided upon. The depth of the water depends to a great extent upon the purpose of the canal. If it is to be an irrigation canal from which many branches may be taken, the depth should be less than for a canal which is meant to carry flood water to a reservoir. For rectangular channels, a canal in which the base is twice the depth gives a maximum velocity for a given gradient. This cross section can sometimes be used in flumes and rock cuts, but it is seldom practicable to construct a canal with the most economical form of trapezoidal section, as the slopes are fixed by the nature of the material, and the depth of water by other conditions.

In general the following limiting values for side slopes of canals will be found safe:

¼ horizontal to 1 vertical in rock cuts.
1 horizontal to 1 vertical in firm soil.
1½ horizontal to 1 vertical in ordinary soils.
2 horizontal to 1 vertical in very light soils.

On steep side hills the channel should be all in excavation, in order to prevent the sliding of the lower bank, although the expense of the grading is thereby increased. In level or slightly rolling country the cost of excavation may be much reduced by making the depth of the cut such as to allow the upper portion of the canal to be made with the material taken out of the excavation.

Due allowance must be made for the height of the banks above the maximum height of water surface, and also for settlement or shrinkage of banks, dependent upon the nature of the material excavated. Great judgment must be used to avoid slides on the lower side of the canal, where the ground on which the embankment rests is sloping, as water is liable to percolate between the new material and the original surface of the ground. Slides are also liable on the upper side of canals on steep slopes if the ground becomes saturated from rain or snow.

Sometimes a berm of 2 or more feet is introduced between the foot of the bank and the edge of the excavation, but this is liable to introduce irregularity in the flow of the water if the ground is sloping laterally and undulating longitudinally. Absolutely level ground is seldom found in practice, and if a berm is graded off to true planes, in order to make the cross section conform as nearly as possible to the theoretical one, the additional cost is not justified. Berms have been very generally discarded in recent irrigation practice, and are not recommended except possibly in deep cuts to catch material that would slide into the canal.

It is nearly always desirable to have a wagon road running along-

side a canal, to facilitate repairs, and this must also be taken into account in the formation of the bank on one side.

In fixing the minimum radius of curvature to be used on a canal it is well to bear in mind that curves may be used much more freely to save work on a canal than they may on railroads. The minimum radius to be adopted must be, in great measure, a matter of good judgment on the part of the engineer. Every curve changes the conditions of flow to some extent, tending to reduce the velocity. Care must also be exercised to prevent excessive washing of the banks. In general, the minimum radius should be from 4 to 6 times the bottom width. In special cases a smaller radius may be used, but then the outer bank should be riprapped in ordinary soils to prevent erosion.

These are a few of the points which must be considered when making the paper location on the topographic sheets. When staking out the final location on the ground the method used in staking out the final location of a railroad should be adopted. Stakes should be set every 50 or 100 feet, and driven so that if struck hard enough they will break off in the ground instead of being knocked over. In hard ground it may be necessary to use a steel gad and sledge to make a hole for the stake, but extra care should be taken with the final location stakes, as every one of them will have to be found and verified by the party setting out the slope stakes. This rule applies even more forcibly to slope stakes, as they are supposed to be preserved by the construction force and should not be disturbed. They should remain just where driven till the work is all done. On many railroads small hardwood pegs are driven exactly to measurement on center and down to the surface of the ground, and the usual large stake driven alongside as a witness stake. This practice has much to commend it. As the center stakes are usually removed after all instrument points are referenced, and the first plowing made, it is sufficient to drive the center stakes so that they may be found without trouble at any time within a few years after being driven.

The stakes of all preliminary lines should be pulled up when the final location stakes are driven. Neglect of this apparently simple precaution will frequently result in the graders getting on the wrong line and doing work which has to be thrown away, thereby causing hard feelings between engineer and contractor. All stakes look alike to a man who can not read. Stakes should always be lettered and numbered on the side next the transitman, so that he can check the numbering on curves. This also enables the rodman to see the number and call it out to the leveler before he puts down his rod. The letter "L" should be used only for final location. All stakes should be lettered and numbered from the top downward and be marked with red keel if possible to obtain it.

Hubs with tack points for beginnings and ends of curves should be

driven to the level of the ground or a little below it, so that passing vehicles or animals may not displace them.

Bench marks should be established on final location at least every 1,000 feet on rough sidehill ground, and at least every mile in level or open country, and they should always be far enough to one side to be well outside the slope stakes when they are set.

DESIGNS.

When the engineer is making the paper location he will decide as to the number and nature of the structures required on the canal, and will have additional topographical notes and maps made on a large scale, as may be necessary for the careful designing of the structures. These designs could be underway when the final location of the line is commenced in the field. The designs should contemplate permanent and durable structures, such as will reflect credit on the service and be most economical in the end.

The engineer in charge of a project should make his own designs for structures or have them made by a competent designing engineer under his immediate supervision, and when ready they should be submitted to the board of consulting engineers appointed by the chief engineer for their approval and suggestions.

When the plans are approved by this board they may be presented to the chief engineer for his final approval, after which they will be ready to attach to the contract.

POWER DEVELOPMENT.

By H. A. STORRS.

The power features of most projects naturally divide themselves into three classes, viz:

1. Temporary power development for use during construction of dams, canals, etc.
2. Permanent power development possibilities.
3. Permanent power utilization possibilities.

Machinery required for temporary power plants can ordinarily be utilized afterwards in the permanent power plants; hence temporary power development may often be accomplished at the slight increase of cost of the project as a whole, since the first cost of most of the machinery is ultimately charged to the permanent plant. To secure this end the general design of the permanent power plant should be determined before buying the machinery for the temporary plant.

For gravity irrigation systems, including the development and use of power for pumping as an auxiliary source of supply, two classes should be considered, viz:

(1) Those in which the power is developed by water flowing out of a storage reservoir.

(2) Those in which the power is developed by water diverted from a stream or falling from a higher to a lower canal level. Power plants for the first class will have to be designed to operate under various heads, depending upon the stage of water in the reservoir; whereas power plants for the second class will be designed to operate under practically constant head, and will therefore be cheaper and simpler in construction.

For each class the available flow of water during the irrigation season should be stated. This information could be best presented in the form of curves showing the probable average day and night flow per day or week throughout the year.

An important aid in determining the best design of power machinery for class 1 consists in knowing approximately the stage of water in the reservoir at which the full output of the power plant is to be secured. The bearing of this information in determining the design becomes evident when it is observed that if the machinery is intended to yield the full output of the plant when the reservoir is nearly empty, the machinery will not be able to utilize the entire power available when the reservoir is full; whereas if the machinery is designed to yield its full output only when the reservoir is full, the plant will be able to deliver only a fraction of its full-rated output when the reservoir is only partially full. Another form of conveying the information required for properly designing power plants for storage reservoirs would involve statements as to capacities at all depths of water in the reservoir and as to whether the reservoir is expected to be filled and emptied each irrigation season, or to be filled and emptied only in exceptional seasons, or to be only partially filled and not entirely emptied (probable limits to be stated) each season under usual conditions of rainfall, etc.

The design and cost of power plants will also be affected by the conditions under which the power is to be used. Assuming that it is to be applied to pumping from wells or from a lower to a higher canal level, the power will usually have to be transmitted electrically, and the distance of transmission is one factor in determining the design of the electrical apparatus. The cost of transmission lines will depend upon the distance, the amount of power transmitted, and local conditions.

In preliminary estimates of cost of power plants, transmission lines, and pumping plants, the knowledge of local conditions which it is most necessary to have involves a statement as to the supply of labor and construction materials afforded by the local market, with estimates of cost of the same, a statement of the distance by wagon road from the nearest railroad station to the location of the works, and cost of hauling per ton mile, and a statement of the nature of the country over which the transmission line will pass. The cost of poles, 35 to 40 feet

long and 10 inches in diameter at the top, delivered, should be ascertained, as well as the cost of timber and other construction materials.

Detailed information should be obtained as to the nature of the ground at the proposed locations of power and pumping plants, preferably on plane-table sheets drawn to a scale of about 50 feet to the inch and showing 5-foot contours over the area where headworks, penstocks, power machinery, draft tube pits, tailrace, etc., are to be located. Where it is expected that structures requiring solid foundations will be built, the depth to bed rock should be ascertained by test pits or borings. The highest and lowest water levels in the stream at proposed location of tailrace should be stated, in order that the power house may be built out of reach of flood waters, yet low enough to utilize the full head available. The power-room floor level can be 18 or 20 feet above tail-water level.

The power possibilities of a project should usually be given but little attention when an ample supply of water can be delivered by gravity at reasonable cost to all the irrigable land available. Sometimes, however, a long main canal through country involving expensive construction may be replaced by a series of canals located so as to take advantage of the surface formations and involving one or more pumping plants, thus lessening somewhat the total cost of the system. A given quantity of water falling a given vertical distance may be counted on to pump an equal quantity of water to about one-half the same vertical height, and the total cost of the electric power system may be roughly figured at $100 per horsepower transmitted.

Projects in which pumping may be considered an essential feature involve three principal items: (1) The possibility of developing power at reasonable cost; (2) an adequate supply of water for pumping; (3) a sufficient quantity of irrigable land within reach of the pumping plants.

The determination of the possibility of power development involves a study of water power from streams and reservoirs and a study of the fuel resources of the country.

The determination of the possibility of obtaining an adequate water supply involves a study of stream flow and underground supply.

The determination of the amount of land to be irrigated by pumped water involves a study of the character and quantities of land at various elevations and at various distances from the source from which water is to be obtained by pumping.

The possibility of storing the pumped water in reservoirs at such elevations that it can be distributed by gravity to the lands to be irrigated is an important feature of some projects. Where such an arrangement is possible, a comparatively small pumping plant working continuously would be able to furnish water sufficient for a short irrigation season for several times as much land as it could supply if pumping direct onto the land without having the benefit of the equal-

izing effect of the reservoir. This feature is specially important where the water is pumped from wells, since a steady draft on the wells is more economical of power than a short forced draft, and the capacity and number of wells required to yield the year's supply of water may be smaller.

Where conditions are favorable for pumping water for irrigation, except that the cost of developing water power is nearly or quite prohibitive, the possibility of using steam or gas engines should be considered. This involves securing information regarding the cost of coal or oil delivered at the power plant, and a determination of the quality of the coal in case a low-grade coal can be obtained cheaply near at hand. With cheap fuel a central producer gas plant distributing gas under pressure by pipe lines 5 or 10 miles in length to scattered pumping plants operated by gas engines would compare favorably, as regards first cost, operating expenses, and efficiency, with many electric systems operated by water power.

PERSONNEL OF THE RECLAMATION SERVICE.

[Revised to April 1, 1904.]

The following list gives, in alphabetic order, the names of the principal men of the regular force of the reclamation service and the salient facts concerning their education and experience. These have been brought together for convenience of reference by all concerned with the work of the reclamation service, since they answer questions that are asked from time to time as to the various men.

More elaborate and detailed information is kept on file in the Washington office, together with confidential records of the ability displayed and the work accomplished by each employee, these being for the information of the boards convened to consider personnel.

AHERN, JEREMIAH, engineer. Address, Washington, D. C. Born in California, July 5, 1860. Educated as a civil engineer at University of California. In 1883 was field assistant with United States Geological Survey, and in November of that year was appointed assistant topographer. In 1884 was on survey and reconnaissance work in California. During field seasons 1885–1889 was on topographic surveys in Virginia, Alabama, Georgia, and Pennsylvania, and during the winter months on office work in Washington, D. C. 1889–1891, topographer, Powell Irrigation Survey, in charge of surveying parties in Montana. 1891–1892, topographer, United States Geological Survey, on topographic work in New Mexico and California. 1892–1895, associated with McCray Brothers, civil and hydraulic engineers, San Francisco, Cal., during which time was engaged in land surveys, surveys of reservoir sites, railroads, canal lines, construction of ditches, and wagon roads, also in hydraulic and irrigation surveys and investigations and study of problems related to water supply. Author of "Hydraulic Formulas and Tables." 1895–1899, United States surveyor of lands in Indian Territory, under United States Geological Survey. 1899–1901, topographer, United States Geological Survey, on topographic work in Arizona, New Mexico, and Utah, and on office work in Washington, D. C. 1901–1902, hydrographer, United States Geological Survey, on topographic work in Colorado and Arizona, and on hydrographic work in Arizona. July 8, 1902, appointed engineer in the reclamation service, since which date he has been engaged in surveys and investigations in Wyoming.

ANDERSEN, CHRISTIAN, engineer. Address, Portland, Oreg. Born in Denmark, December 28, 1860. Educated at University of Minnesota, degree B.C.E. Member American Society Civil Engineers. In 1886 was division engineer on construction for Minneapolis, Lyndale and Minnetonka Railroad; in 1887, resident engineer on construction for Northern Pacific Railroad; 1888–1890, topographer, transitman, and in charge of locating party, Oregon Railway and Navigation Company. From 1890–1899 practiced civil engineering in Oregon. In 1899–1900, chief engineer of location, The Dalles Southern Railroad. In 1901 was general manager for the Columbia River Packers' Association, constructing salmon cannery in Alaska. In 1902, superintendent of construction in Alaska. Appointed assistant engineer in the reclamation service May 24, 1903, and promoted to engineer January 1, 1904.

ARNOT, EDWIN PERCY, engineering aid. Address, Wadsworth, Nev. Born in California, July 22, 1882. Graduate of Nevada State University, degree B.S. In 1902 was with Standard Electric Company, of California, making estimates of rock-fill in construction of dam at Tamarack, Cal., also making topographic survey of two reservoir sites. 1902–1903, with United States Geological Survey, on Truckee Canal and Carson River reservoir sites. May 27, 1903, appointed engineering aid in the reclamation service, and continued on Truckee-Carson work.

BABB, CYRUS CATES, engineer. Address, Washington, D. C. Born in Portland, Me., 1867. Graduate of Massachusetts Institute of Technology, 1890. Associate member American Society Civil Engineers. In 1887, on preliminary railroad survey Rockland to Rockport, Me. In 1888 transitman under Gilbert Hodges, Member American Society Civil Engineers, on street railway work; 1890 to date with United States Geological Survey in various capacities, viz., July, 1890, to August, 1892, assistant hydrographer and assistant topographer in charge of hydrographic office work; August, 1892, to January, 1893, assistant in topographic and hydrographic field work in Kansas, Nebraska, and Colorado; from January to July, 1893, in office; summer and fall of 1893, assistant geologist working in Georgia, Alabama, and Tennessee; summer of 1894 traveling in Europe; January to June, 1895, engaged in hydrographic field work in the basin of Potomac River; July to December, 1895, in charge of hydrographic work in Atlantic slope; January to July, 1896, as assistant hydrographer for the determination of a water supply for Gila River Indian Reservation in Arizona; July, 1896, to November, 1898, inspector of hydrographic stations west of Mississippi River. November, 1898, to September, 1899, assistant hydrographer in charge of field party on Gila River survey; September, 1899, to June, 1901, hydrographer in charge of investigations for water supply for Uinta Indian Reservation, Utah; July, 1901, to June, 1902, hydrographer in charge of Milk River project, Montana. July, 1902, to date, engineer, reclamation service, in charge of Milk River project, Montana.

BAILEY, EARNEST ALBERT, engineering aid. Born at Clay Center, Kans., November 9, 1879. Education, high school, Delta, Colo., and University of California. In 1902, engaged in land and general surveying in Mendocino County, Cal. Appointed engineering aid in the reclamation service May 15, 1903.

BARROWS, HAROLD KILBRETH, assistant engineer. Address, Burlington, Vt. Born in Massachusetts, November 9, 1873. Graduate of Massachusetts Institute of Technology, 1895, in civil engineering. In 1895 and 1896 was assistant instructor in civil engineering at Massachusetts Institute of Technology. 1896–1899, with city engineer, Newton, Mass., as transitman and assistant in charge on city work. 1899–1901 with Metropolitan Water Board, Boston, Mass., on designs and computations. 1901–1903, assistant professor of civil engineering at University of Vermont, and from that time associate professor. July to September, 1902, resident hydrographer, United States Geological Survey, Kingston, N. Y. Was appointed assistant engineer June 15, 1903, in charge of river work in Vermont and New Hampshire, under N. C. Grover.

BEBB, EDWARD CROSBY, topographer. Address, Seward, Ill. Born in Illinois 1872. High school education at Rockford, Ill., and course in civil engineering at University of Wisconsin, B.S. in C.E., 1896. In 1896 levelman and topographic assistant with United States Geological Survey in northern peninsula of Michigan. 1896–1897, in general engineering work, making survey and plat for addition to city of Madison, Wis., gaging Baraboo River, etc., and preparing maps and drawings for Geological Survey publications. July 1, 1897, appointed assistant topographer United States Geological Survey and assigned to duty with parties in Vermilion iron range, Minnesota, and on Danville, Ill., quadrangle. In 1898 in charge of topographic work in Menominee iron range, Michigan, and later in Vermilion range, Minnesota. July 1, 1899, appointed topographer United States Geological Survey. 1899–1903, in charge of topographic work in various mining districts and surveys of quadrangles. July 1, 1903, transferred as topographer to reclamation service.

BEERS, WILLIAM DUKE, assistant engineer. Address, P. O. Box "S," Salt Lake City, Utah. Born in Utah, August 20, 1877. Graduate of Agricultural College of Utah, degree B.S. In 1897 engaged in conduct of experiments on carrying capacity of ditches and canals. In 1899 engaged in running canal lines and gaging streams. In 1900 engaged in preliminary and location work on proposed Idaho Southern Railroad. In 1901 worked under Mr. G. L. Swendsen as assistant field engineer on the construction of the Last Chance canal in Idaho. 1901–1902, assistant civil and irrigation engineer at Agricultural College of Utah. 1901–1903, assisting in hydrographic work in Utah. February 7, 1903, appointed engineering aid in the reclamation service; January 1, 1904, promoted to assistant engineer.

BENNETT, SAMUEL G., engineer. Address, 431 Rialto Building, San Francisco, Cal. Education, public and private schools, Hartford Academy, Hartford, Ky., and Georgetown College, Georgetown, Ky. From 1886 to 1891 assisted in surveying and engineering work; 1891–1893, draftsman; 1893–1896, general practice, surveying, and engineering. In 1897 assisted in survey of Music Mountain mines, Arizona. 1897–1898 with Chicala Water Company, Rialto, Cal. April to October, 1898, in general surveying work. 1898–1903, assistant resident hydrographer, United States Geological Survey. 1899–1900, conducted investigations and made estimates in connection with water supply, pressure, etc., for Pasadena and Los Angeles, Cal. July, 1902, made a study of carrying capacity of the Lagrange Ditch and Hydraulic Mining Company's canal, Lagrange, Cal. Appointed engineer in the reclamation service January 8, 1903.

BIEN, MORRIS, engineer. Address, Washington, D. C. Born in New York City, April 17, 1859. Graduate of University of California in civil engineering, Ph. B., 1879; graduate of Columbian University, Washington, D. C., LL.B., 1895; National University, Washington, D. C., LL.M., 1896. 1879–1893, engaged in topographic and irrigation work, United States Geological Survey. 1893–1902, General Land Office, Washington, D. C., legal and right-of-way work relating to irrigation, railroads, etc. July 10, 1902, transferred to Geological Survey; engaged in legal and land matters, reclamation service.

BLANCHARD, CLARENCE JOHN. Address, Washington, D. C. Born, Norham, Canada, December 4, 1863. Education, Iowa Agricultural College, class 1883. Deputy State oil inspector, Iowa, 1886–1891. Special press correspondent in Central America 1893–1898; in Hawaii 1899. Chief of irrigation section Twelfth Census. In charge of special investigation of irrigation by Census Office 1902-3. March 1, 1903, appointed clerk in the reclamation service.

BLISS, GEORGE HENRY, assistant engineer. Address 427 Peyton Block, Spokane, Wash. Born at Newburyport, Mass., April 21, 1875. Education, high school and Massachusetts Institute of Technology. 1894–95, engineering inspector, etc., for city of Newburyport, Mass. 1896–97, assistant engineer for city of Malden, Mass. January–April, 1898, engaged in construction work on electric road, Boston, Mass. April–October, 1898, with Allegheny and Western Railroad, in charge of party railroad location surveys. October, 1898–1900, with New York, New Haven and Hartford Railroad, Boston, track widening, retaining wall, and bridge work. 1900–1902, inspector of materials, Charlestown Navy-Yard. 1902–3, with Boston Steel and Iron Company, Medford, Mass.; structural engineering. Appointed assistant engineer in reclamation service, May 23, 1903.

BOLSTER, ROY HALE, assistant engineer. Address Phoenix, Ariz. Born Roxbury, Mass., April 6, 1877. Studied electrical and civil engineering at Massachusetts Institute of Technology. Appointed engineering aid in the reclamation service January 19, 1903, and from that date to April 29 on duty in Washington, D. C., making computations. May–October, 1903, at Montrose, Colo., on Gunnison River project, and then in electrical work in connection with Salt River project. January 1, 1904, promoted to assistant engineer.

BONSTEDT, FERDINAND, assistant engineer. Address, Lawton, Okla. Born Akron, Ohio, 1864. B. S. in C. E. Pennsylvania Polytechnic College, 1885. July, 1885, to January, 1886, employed in city engineer's office, Akron, Ohio. 1886, transitman on location and division engineer on construction Union Pacific Railway extension in Nebraska. 1887–1889, assistant engineer municipal improvements for city of Lincoln, Nebr. 1890, engineer on maintenance of way for Burlington and Missouri River Railroad. 1891–1896, deputy city engineer, Lincoln, Nebr. 1896–1900, city engineer, Lincoln, Nebr. 1900–1902, assistant engineer, department of streets, Habana, Cuba. Appointed assistant engineer United States Geological Survey, September 29, 1902.

BRAINARD, LINUS BROOKS, engineering aid. Address, Los Angeles, Cal. Born in Oregon, June 20, 1879. Graduate of University of California, degree A. B. June–August, 1901, solar compass work, subdividing two townships, 135 miles of line. June–November, 1902, field work, Department of Agriculture, running about 500 miles of levels, and having immediate supervision of another party doing similar work; later employed on office work. May 15, 1903, appointed engineering aid in the reclamation service.

BRANCH, LESTER VAN NOY, assistant engineer. Address, Chamber of Commerce Building, Denver, Colo. Born in New York State, February 23, 1880. Graduate of University of North Carolina; degree B. S., and, later, M. S. Took special course at Columbian University, 1901–1903. With United States Geological Survey June–November, 1900, on Appalachian Park investigations in North Carolina and Tennessee. November, 1900, to May, 1901, in office at Washington on work connected with Appalachian Park investigations and on computations for annual report. May–November, 1901, engineering aid in Utah and Montana. November, 1901, to July, 1902, on office work in Washington office. July 28, 1902, appointed assistant irrigation engineer in the reclamation service, and has since been on duty in Montana, Colorado, and Wyoming, and in Washington office.

BRINKLEY, MILO HAMILTON, engineering aid. Address, Jefferson City, Mo. Born in Missouri February 12, 1881. Graduate of University of Missouri, B. S. in C. E. In 1901 was chainman on a preliminary railroad survey in Missouri. In 1902 was with the Gulf and Ship Island Railroad Company, of Mississippi, at first as rodman and afterwards on instrument work; also as draftsman in the office of the chief engineer. June 10, 1903, was appointed engineering aid in the reclamation service, and was transitman on Bear Lake project in Idaho until October.

BURKE, JOHN THOMAS, assistant engineer. Address, Boise, Idaho. Born in Illinois September 28, 1874. Graduate of Idaho State University, B. S. in C. E., 1901. In 1900 was assistant draftsman and surveyor on aerial tramway work in Nelson, British Columbia. In 1901 topographer on preliminary canal lines for Twin Falls Land and Water Company; also, in same year, topographer on reservoir and canal investigations for United States Geological Survey in Idaho. 1901–2, assistant engineer and superintendent on reservoir and canal construction. In 1902 assistant draftsman and field engineer under Idaho State engineer. 1902–3 assistant irrigation engineer in charge of field party for United States Geological Survey on reservoir and canal investigations in Idaho. January 12, 1903, appointed assistant engineer in the reclamation service and assigned to Colorado River surveys near Needles, Cal., and later in charge of field work on surveys connected with Minidoka project in Idaho.

CAMERON, JOHN DONALD, engineering aid. Address, Reno, Nev. Born in Nevada June 11, 1881. Education, high school and University of Nevada, degree B. S. In 1903 was in the field with a topographic party engaged in surveying a proposed canal line, also in same year engaged in transit work near Stillwater, Nev. In fall and winter of 1902 in office of L. H. Taylor, resident engineer, United States Geological Survey. May 23, 1903, appointed engineering aid in reclamation service and assigned to duty as levelman on location work of Lower Truckee canal from April to July,

inclusive; then as transitman on location of laterals tributary to Lower Truckee canal; also on office work in Reno, Nev.

CAMP, JAMES G., assistant engineer. Address, Phoenix, Ariz. Born, Alliance, Ohio, July 2, 1863. Education, common and high schools. February, 1894, to May, 1898, superintendent of construction for Farmers' Union Ditch Company. June–November, 1898, topographer with Pacific and Idaho Northern Railroad Company. From May, 1902, with United States Geological Survey party, making preliminary reservoir surveys, etc. January 12, 1903, appointed engineering aid in the reclamation service, and promoted to assistant engineer September 1, 1903. January–May, 1903, preliminary surveys on Umatilla River for ditch and reservoirs and investigation as to feasibility of irrigation projects on the Palouse, Snake, and Columbia rivers. June–November, 1903, preliminary surveys and investigations of ditch projects and reservoirs on North Fork of Snake River.

CASTEEL, CALVIN, engineering aid. Address, Salem, Oreg. Born in Iowa, January 20, 1876. Graduate of University of Oregon, B. S. in C. E. June 16, 1903, appointed engineering aid in the reclamation service, and assigned to duty in the State of Washington.

CHANDLER, ALBERT EDWARD, engineer. Address, Carson, Nev. Born at San Francisco, Cal. Graduate of University of California. 1899–1902, instructor at University of California. 1900, hydrographer United States Geological Survey, in charge of investigations of storage possibilities on Cache Creek, California. 1901, irrigation expert, United States Department of Agriculture, in charge of study of duty of water on Tule River, California. 1902–3, irrigation assistant, Department of Agriculture, in charge of irrigation investigations on Carson and Walker rivers, Nevada. Appointed engineer in the reclamation service June 26, 1903.

CHANDLER, ELWYN FRANCIS, assistant engineer. Address, University, N. Dak. Born in Ohio, August 29, 1872. Graduate of Ripon College, Wisconsin, degrees A. B. and A. M. Attended University of Wisconsin two years. Has been engaged in surveying and civil engineering since 1890. 1895 was city engineer of Ripon, Wis.; in 1898 was with Wisconsin Geological and Natural History Survey, on a hydrographic map of Green Lake, Wisconsin. 1899, employed in designing and constructing a sewerage and drainage system for University of North Dakota, and other similar improvements there. In 1902 was in charge of an office in Grand Forks, N. Dak., for general civil engineering and surveying. 1902–3, assistant professor in charge of the department of mathematics, University of North Dakota. Appointed assistant engineer in the reclamation service May 1, 1903.

CHARLES, LAVERN JOHN, assistant engineer. Address, Golden, Colo. Born in Colorado, July 9, 1877. Education, common school, high school, and technical; degree of engineer of mines and metallurgy. Technical course included survey of reservoir sites, construction of irrigation canals, testing and rating flow of water over weirs, through conduits, pipes, ditches, etc., tunneling and underground surveying, and general hydrographic surveying. Appointed assistant engineer in reclamation service March 31, 1903, since which date has been engaged in measurement of base lines, triangulation, transit, level, and plane-table work, and computations incident thereto.

CHILDS, ERNEST ROY, assistant engineer. Address, 2045 J street, San Diego, Cal. Born in Minnesota, July 7, 1875. Education, University of California, degree B. S. 1901–1903, field assistant with United States Geological Survey. Appointed engineering aid in the reclamation service July 1, 1903; promoted to assistant engineer January 1, 1904.

CHURCHILL, PERCIVAL MITCHELL, assistant hydrographer. Address, Elmwood, Mass. Born at Elmwood, Mass. Education, full civil engineering course at Massachusetts Institute of Technology, class of 1895. In 1892 was in city engineer's office, Brockton, Mass. 1893, draftsman, East Bridgewater, Mass. 1894, private

work at same locality. 1895–96, resident engineer, Massachusetts highway commission. 1896–97, assistant chief of party for joint board of Massachusetts State board of health and harbor and land commission on survey of Green Harbor, Mass. 1898, draftsman and designer, East Bridgewater, Mass. 1899, chief of party on electric railway work, Boston. 1900–2, United States junior engineer, War Department. 1902, resident hydrographer United States Geological Survey, Catskill Mountain district, and Delaware River. Appointed assistant hydrographer, reclamation service. December 10, 1902.

CLAPP, SIDNEY K., assistant engineer. Born in Massachusetts, January 8, 1873. Graduate Massachusetts Institute of Technology, B. S. In 1894, in engineering department of city of Boston, Mass. 1895, with Boston and Maine Railroad. 1895–1901, with Metropolitan Water and Sewerage Board of Massachusetts. 1901–2, under city engineer, Waterbury, Conn., on Wigwam dam. 1902–1904, in private employment. January 5, 1904, appointed assistant engineer in the reclamation service.

CLAPP, WILLIAM BILLINGS, assistant engineer. Address, Pasadena, Cal. Born at Conway, Mass., April 11, 1861. Education, high school and special study. 1882, assistant in subdivision of public land in northern California. 1883–1886, assistant engineer in charge of construction, Los Angeles and San Gabriel Valley Railroad. 1888, assistant engineer on construction of cable railway system, also assistant engineer for Los Angeles and Salt Lake Railway Company. 1889, private engineering work at Seattle, Wash. 1891, engineer and roadmaster for San Gabriel Valley Rapid Transit Company. 1892, chief engineer Arizona waterworks and sewerage company. 1893, engineering practice, Pasadena, Cal. 1894–1901, city engineer, Pasadena, Cal. 1901–1903, private engineering practice and with United States Geological Survey. Appointed assistant engineer in reclamation service May 5, 1903.

CLAUSEN, JACOB CENTENNIAL, engineer. Address 314 Fourth street, Los Angeles, Cal. Born at Blanco, Cal., March 8, 1876. . Graduate of University of California, B. S. In 1898 was with city engineer of Salinas City, Cal. 1899, with county surveyor, Monterey County, Cal. 1899–1901, surveying for De Lamar Mining Company (Limited), Idaho. 1901, in charge of transit party for county surveyor, Monterey County, Cal. 1901–2, with Spreckels Sugar Company, surveying, estimating, and locating in connection with Arroyo Seco canal, reservoir, and dam, impounding waters of San Lorenzo Creek, etc.; also in connection with distribution of water, making contour surveys, locating lateral ditches, checks, levees, gates, check feeders, etc., also measuring quantity of water passing through gates. 1903, with United States Geological Survey in hydraulic and hydrographic work. May 6, 1903, appointed assistant engineer in the reclamation service; January 1, 1904, promoted to engineer.

CORLETT, BERTRAM EDWIN, engineering aid. Address, Reno, Nev. Born at Cleveland, Ohio, March 18, 1879. B. S., University of California. With surveying party from University of California two summers, performing transit, level, stadia, and plane-table work. June 1, 1903, appointed engineering aid in reclamation service.

COVERT, CLERMONT CALVERT, assistant engineer. Address, Binghamton, N. Y. Born at Townsendville, N. Y., January 25, 1872. C. E., Ohio Northern University. In 1895 was in general practice at Syracuse, N. Y. In 1896, with city engineer, Syracuse, N. Y. 1896–1898, in private practice at Binghamton, N. Y. 1898–99, with United States Volunteer Engineers in Porto Rico; also made preliminary survey for Penn Yan, Lake Kenka and Southern Railroad. 1899–1900, with Choctaw, Oklahoma and Gulf Railroad. 1900–1, in private practice. 1901–2, assistant engineer in charge of construction work Delaware, Lackawanna and Western Railroad. 1902–3, assistant hydrographer, United States Geological Survey. June 1, 1903, appointed assistant engineer in the reclamation service.

COX, CHARLES BARROWS, engineering aid. Address, 106 Washington street, Newton, Mass. Born in Massachusetts November 4, 1881. University education; graduate of Massachusetts Institute of Technology, degree S. B., 1903. In 1901, with

Holbrook, Cabot & Daly on New York City subway, general office work, leveling, etc. In 1902, Summer School of Massachusetts Institute of Technology, gaging streams, plane-table mapping, etc. June 30, 1903, appointed engineering aid in the reclamation service.

DARTON, NELSON HORATIO, geologist. Address, Washington, D. C. Born in New York City, December 25, 1865. Received a common school education. In 1878 entered pharmaceutical laboratory of William W. Thayer, in New York City. In 1882 opened a laboratory as analytical chemist in New York and gave much time to research work on methods of chemical analyses, the results of which were published by the American Chemical Society, of which he was elected a member, in 1883. First geological studies were undertaken in New Jersey, near New York City, where observations of the Newark group were made, which finally developed into a detailed investigation. In 1886 joined the United States Geological Survey, and has been a member of the Survey's geological corps ever since. Mr. Darton's earlier work was mainly in New Jersey, New York, Maryland, Virginia, and West Virginia, but during later years most of his attention has been given to the Black Hills, central Great Plains, and Big Horn Mountains. In the spring of 1903 was placed in charge of the western section of hydrology of the hydrographic branch of the Survey to investigate the geology of underground waters in the reclamation States and Texas, with a number of assistants. The Survey has published many reports and folios by Mr. Darton, and numerous papers of his have appeared in scientific journals and the proceedings of scientific societies.

DAVIES, WILLIAM GOMER, engineering aid. Address, Los Angeles, Cal. Education, Iowa State College of Agriculture and Mechanic Arts; Pomona College, Claremont, Cal.; University of California, degree B. S. In 1901 was with Government irrigation survey on Tule River, California. In 1902 with survey party on Sierra Railroad of California and with Government drainage survey, Fresno, Cal. May 15, 1903, appointed engineering aid in reclamation service.

DAVIS, ARTHUR POWELL, supervising engineer. Address, Washington, D. C. Born in Illinois, February 9, 1861. Education, Junction City, Kans., High School; graduate of State Normal School, Emporia, Kans.; engineering studies, Columbian University, Washington, D. C. 1884–1894, topographer, United States Geological Survey, conducting surveys and explorations in Arizona, New Mexico, and California. 1895–1897, hydrographer in charge of all Government stream measurements. 1898–1900, hydrographer in charge of hydrographic examination of Nicaragua and Panama canal routes. Member American Society of Civil Engineers. Author of Elevation and Stadia Tables, 1893; Progress of Stream Measurements, 1897; Irrigation near Phoenix, Ariz., 1897: Hydrography of Nicaragua, 1899; Irrigation Investigation in Arizona, 1898; Hydrography of the American Isthmus, 1902; Water Storage on Salt River, Arizona, 1903; also articles in magazines on irrigation, the isthmian canals, and other hydrographic subjects. Appointed principal engineer, reclamation service, July 8, 1902, and supervising engineer, July 1, 1903.

DAVIS, EDMUND IRELAND, assistant engineer. Address, Washington, D. C. Born in Maine, November 26, 1877. Education, four years at University of Maine and advanced work in hydraulics at Cornell. In 1900 was with United States Geological Survey; also in same year with Boston and Albany Railroad on preliminary and location surveys. In 1901 with United States Geological Survey, and inspector of surveys, Sullivan, Me. In 1902 with United States Geological Survey, also on plane-table work, etc., at Rockland, Me., and Ithaca, N. Y. May 11, 1903, appointed engineering aid in the reclamation service, and on November 11, 1903, promoted to assistant engineer.

DILLON, RALPH P., assistant engineer. Address, Chamber of Commerce Building, Denver, Colo. Born, Fremont, Ohio, December 14, 1872. Education, common and

high school, graduate of Case School of Applied Science, June, 1894, degree B. S. In 1896 was assistant city engineer of Alexandria, Ind. 1896–97, with Youngstown (Ohio) Bridge Company. 1897–98, with Brown-Ketcham Iron Works, Indianapolis, Ind. 1898–99, with Gillette-Herzog Manufacturing Company, Minneapolis, Minn. 1899–1900, with C. L. Strobel, consulting and contracting engineer, Chicago, Ill. 1900–1902, with Keystone Bridge Works, Pittsburg. 1902–3, with United States Geological Survey, on Colorado River. May 28, 1903, appointed engineering aid in the reclamation service. July 1, 1903, promoted to assistant engineer.

DOBSON, FRANK S., engineering aid. Address, Roswell, N. Mex. Education, three years high school and preparatory school, three and one-half years in University of Nebraska, C. E. course. 1896–1899, employed, while attending school, as chainman, rodman, and instrument man by city engineer, Lincoln, Nebr. In 1900, axman and chainman, Burlington and Missouri River Railroad, in Wyoming. September, 1900, to June, 1901, instrument man in surveying city lots, setting sewer and street grades, Lincoln, Nebr. June–October, 1901, assistant division engineer, Burlington and Missouri River Railroad, Edgemont, S. Dak. From October, 1901, to June, 1902, on river measurements in Nebraska for United States Geological Survey. June–September, 1902, instrument man in Bridgeport, Nebr. September, 1902, general engineering work in Fairbury and York, Nebr. During fall season of 1902, on river measurements for United States Geological Survey. April–May, 1903, instrument man on railroad location work, Burlington and Missouri River Railroad. May 25, 1903, appointed engineering aid in the reclamation service, and has since been in charge of field parties in New Mexico.

DOLE, RICHARD BRYANT, engineering aid. Address, 263 Dearborn street, Chicago, Ill. Born, Portland, Me., May 8, 1880. Graduate of Portland (Me.) High School, classical course, 1898; graduated Bowdoin College, 1902. In 1902 was elected instructor in chemistry in Bowdoin College; resigned in February, 1903, to take post-graduate course at Massachusetts Institute of Technology, specializing in sanitary chemistry and bacteriology. In 1902, worked on water analysis and study of Androscoggin River for United States Geological Survey. June 15, 1903, appointed engineering aid in reclamation service.

DOWNEY, AUSTIN C., engineering aid. Address, Fort Collins, Colo. Born in Illinois, November 8, 1881. Education, Fort Collins, Colo., public schools and Colorado Agricultural College; course in civil and irrigation engineering. During college course was employed by Colorado Experiment Station on seepage measurements and as draftsman. In 1902 was on maintenance of way work on Oregon Short Line Railroad. In 1903, in Colorado State engineer's office and afterwards as draftsman for Arkansas Valley Sugar Beet and Irrigated Land Company. June 28, 1903, appointed engineering aid in the reclamation service.

DURYEE, EDWARD, cement expert. Address, Globe, Ariz. Born in New York State in 1858. Graduated from engineering department, Rutgers College, 1878, degrees B. S. and C. E. Spent one season with New Jersey geodetic survey. In 1879, engaged in experimental investigations and in perfecting the rotary kiln for metallurgical uses, with G. Duryee. 1880–1892, assayer and metallurgist with smelting companies in New Mexico, Colorado, and California. Instructor in New York schools one season. 1892–1894, superintending chemist, Montezuma Portland Cement Company. 1894–1903, chemist California Portland Cement Company. Occasional engagements as cement expert in investigating deposits of cement materials. With reclamation service, as cement expert, since April, 1903.

ELA, EDWIN STANTON, assistant engineer. Address, Washington, D. C. Born in Wisconsin, December 30, 1871. Education, common school, and four years in academy. Graduate of University of Wisconsin, B. S. in C. E. 1896–1898, with United States Mississippi River Commission on topographic and precise-level work. April–June, 1898, with United States Engineers, New Orleans, La. 1898–1899, with United

States Geological Survey in Kansas, Iowa, Missouri, and Illinois, as field assistant, sketching topography. 1900–1901, with Chicago and Northwestern Railway Company in Iowa, as instrument man and in charge of work on second-track construction. 1901–1903, with United States Geological Survey in Pennsylvania, Maryland, West Virginia, and Georgia, as topographer, doing stadia traverse, and sketching topography. March 7, 1901, appointed assistant topographer in United States Geological Survey, and on July 1, 1903, transferred to the reclamation service.

EVANS, HENRY ROSENGARTEN, assistant engineer. Address, Phoenix, Ariz. Born in Pennsylvania, August 3, 1878. Education, common school, high school, and college. In 1899 was elected county surveyor of Jefferson County, Colo. In 1900 was surveyor-general of Colorado; afterwards engineer for Gross Placer Company in Dutch Guiana. In 1902 was instructor in mining engineering at Nevada State University. April 10, 1903, appointed engineering aid in the reclamation service, and on October 10, 1903, promoted to assistant engineer.

EWING, WILLIARD REA, engineering aid. Address, 1142 South Ridgeway avenue, Chicago, Ill. Born in Ohio, February 10, 1882. Educated at Lawndale Grammar School, Chicago, English High and Manual Training School, and Purdue University, degree B. S. in C. E. In 1901 was draftsman for American Car and Foundry Company, Chicago. In 1902, rodman for Chicago and Northwestern Railway Company; also, same year, chainman on location of Lake Bluff cut-off, Chicago and Northwestern Railway. June 25, 1903, appointed engineering aid in the reclamation service.

FARISH, WILLIAM ADAMS, assistant engineer. Address, Phoenix, Ariz. Born San Francisco, Cal., October 22, 1870. In 1889 was with hydrographic division of United States Geological Survey and afterwards transferred to topography. In 1891 surveyed reservoir sites for Rio Verde Canal Company. In 1893 surveyed Tonto reservoir site, Salt River, Arizona. From 1894 to 1903 was manager and engineer of Arizona Waterworks and Sewerage Company, of Phoenix, Ariz. In 1899 planned and built a sewer and water system for United States Indian school near Phoenix, Ariz. 1899–1903, city engineer of Phoenix, Ariz. June, 1898, to December, 1900, in charge of United States Geological Survey gaging station on Salt and Verde rivers. June 1, 1903, appointed assistant engineer in reclamation service.

FELLOWS, ABRAHAM LINCOLN, engineer. Address, Chamber of Commerce building, Denver, Colo. Born in Kennebunk, Me., November 1, 1864. Received degree of B. A. at Yale, 1886. Commenced work in irrigation lines September, 1887, continuing for eight years for the Montezuma Valley irrigation companies, and being for several years of this period superintendent of schools and county surveyor of Montezuma County, Colo. Was for five years deputy State engineer of Colorado. Prior to appointment as engineer in the reclamation service, on January 1, 1903, was employed at various times by the United States Geological Survey as hydrographer and topographer. Author of several papers and articles on hydrography, irrigation survey, and irrigation practice.

FIELD, JOHN ELLIS, engineer. Address, Chamber of Commerce building, Denver, Colo. Born in Colorado, May 13, 1867. Ph. B., Yale University, class of 1888, scientific. From 1888 to 1893 was engaged in practice of mining, civil, and hydraulic engineering in mining camps of Colorado. 1888–89, assayer and chemist for Aspen Public Ore Sampling Works, Denver. 1890–91 in similar capacity with Taylor & Brenton Sampling Works. 1891–1893, civil and hydraulic engineering practice. 1893–1897, chief examiner of mineral surveys for United States surveyor-general of Colorado. 1897–1899, State hydraulic engineer of Colorado. 1899–1901, assistant engineer board of public works, Denver, Colo. 1901–1903, deputy State hydraulic engineer of Colorado. Author of various bulletins published by United States Department of Agriculture. Appointed engineer in the reclamation service, January, 1903.

FISHER, CHESTER CENTENNIAL, engineering aid. Address, Boise, Idaho, care of D. W. Ross. Born in Oregon, July 4, 1876. Education, high schools and University of Oregon, degree B. S. in C. E. From June 10 to August 19, 1902, was in charge of party cruising timber in the coast fir regions. On June 11, 1903, was appointed engineering aid in the reclamation service, since which date has been employed in transit, level, and plane-table work.

FISHER, HOWELL TRACY, assistant engineer. Address, 20 West Thirty-fourth street, New York City. Born December 10, 1868. Graduate Pottsville, Pa., High School. From 1885 to 1886, with real estate company, compiling land maps. 1886–1888, with State geological survey of Pennsylvania anthracite survey. 1888–89, with Susquehanna Coal Company and Pennsylvania Railroad Coal Company. 1889–1891, with A. B. Cochran & Son in civil and mining engineering and city surveying. 1891–1893, assistant to A. D. W. Smith, geologist and mining engineer, preparing Anthracite Coal Waste Commission's report, and State geological survey of Pennsylvania, assisting in construction of geological map of the State. 1893–1898, with A. B. Cochran & Son, as general assistant. 1898–1900, with United States Board of Engineers on Deep Waterways. 1900–1901 with Isthmian Canal Commission. 1902, with Denver Union Water Company, engaged in estimating capacity of Lake Cheesman reservoir and making plans of spillway for Lake Cheesman dam and reservoir surveys. January 1, 1903, appointed assistant engineer in the reclamation service.

FITCH, CHARLES HALL, engineer. Address, Washington, D. C. Born in Wisconsin, September 12, 1854. Education, preparatory school, Woodside, College Station, Md., and Georgetown Academy, D. C. 1872–1876, aid, United States Coast Survey, and at the same time completed course under private instructors. 1876–1884, United States deputy surveyor on public land surveys in New Mexico, also deputy mineral surveyor and draftsman, surveyor-general's office, Santa Fe, N. Mex. In 1884, appointed topographer, United States Geological Survey. 1884–1888, on topographic surveys in Texas; 1889, on similar surveys in Colorado. In 1890, in California making investigations of alleged fraudulent surveys, under direction of Commissioner of General Land Office. In 1891–92, on topographic surveys in Black Hills, South Dakota. 1893–1895, in General Land Office, Washington, D. C., as assistant chief of surveying division. In 1895, reappointed topographer, Geological Survey, and designated to take charge of topographic and public land surveys of Indian Territory; completed field work in 1898, and office work in 1899. In 1899–1900, engaged in examination of forest reserves in California. In 1900 was transferred to division of hydrography, Geological Survey, and assigned to hydrographic investigations in Northwestern States during 1900–1901. In 1902, appointed engineer in the reclamation service.

FORBES, BURT E., assistant engineer. Address, Beatrice, Nebr. Born in Michigan, August 6, 1872. Graduate of University of Nebraska, 1895, liberal arts course. 1889–90, rodman and chainman, Beatrice, Nebr. 1896–97, teacher, Beatrice High School. 1897–98, levelman on placer mining work in Boulder County, Colo. 1899, engineer in charge of same work. 1899–1901, assistant engineer on railroad construction and track laying. 1901–2, assistant secretary of State board of irrigation of Nebraska, on steam gaging and adjudication of water rights. November 1, 1902, appointed assistant engineer in reclamation service. 1902–3, on plane-table work in Nevada. 1903, transitman on final location and division engineer on construction of main canal, Truckee project, Nevada.

FRENCH, JAMES ADAMS, assistant engineer. Address, Washington, D. C. Born, Washington, D. C., January 27, 1866. Education, common school, and Georgetown University, Washington, D. C. In 1883 was with United States Coast and Geodetic Survey on Long Island Sound. Remained in Coast-Survey service until 1886. Then entered employ of Greely, Carlson & Co., civil engineers, Chicago. 1886, draftsman, Chicago, Burlington and Quincy Railroad Company. 1886–87, instrument

man, maintenance of way department, Union Pacific Railroad, Denver, Colo. 1888, with city engineer of Santa Barbara, Cal. In fall of same year with United States Coast and Geodetic Survey in San Francisco. 1889–1891, in Alaska with party to determine crossing of one hundred and forty-first meridian, on Yukon and Porcupine rivers, also topographic surveys. 1891, Stockton, Cal., with city engineer. 1892, North Yakima, Wash., topographic work for Sunnyside Canal. 1892–93, with Colorado Irrigation Company, Yuma, Ariz. 1893, inspector in engineer department, Washington, D. C. 1900, Cape Nome, Alaska, in charge of equipment of party. 1893–1903, employed chiefly on construction work of sewers, and foundations for sewerage pumping stations, in engineer department of District of Columbia. January 5, 1903, appointed assistant engineer in reclamation service.

FULLER, MYRON LESLIE, assistant geologist. Address, Washington, D. C. Born at Brockton, Mass., April 19, 1873. Graduate of Massachusetts Institute of Technology, geological department; degree, S. B. In 1896 was engaged in geologic work on Cape Cod, a report on which received the Walker prize of the Boston Society of Natural History for 1897. From 1897 to 1900 was assistant and instructor at the college named, and in 1899 also acted as special agent for the commission to the Paris Exposition, preparing an exhibit of building and ornamental stones, which was awarded a gold medal. In June, 1900, joined the United States Geological Survey, since his connection with which has been engaged on geologic surveys in New York, Pennsylvania, Indiana, and Illinois, and has prepared and published several folios and a number of papers on oil and gas. January 1, 1903, was transferred to the hydrographic branch, where he organized the eastern section of the division of hydrology, under which work was begun in about twenty States.

GAY, LEON LINCOLN, engineering aid. Address, Phoenix, Ariz. Born, Barton Landing, Vt., May 29, 1879. Graduate of Sheffield Scientific School of Yale, 1901, Bachelor of Philosophy. 1901–1903, with Boise-Payette River Electric Power Company, Boise, Idaho, as assistant engineer. Construction of canal and power plant began in January, 1902, and from this time until its completion in February, 1903, was engineer in charge. May 13, 1903, appointed engineering aid in the reclamation service.

GILBERT, ARCHIBALD MARVINE, engineering aid. Address, Washington, D. C. Born in Washington, D. C., December 6, 1876. Education, common school, grammar school, and Cornell University. During the summers of 1900 and 1902 was with United States Geological Survey on stream measurement work, plane-table work, etc. June 19, 1903, appointed engineering aid in the reclamation service.

GILES, JAMES MARVIN, assistant engineer. Address, Temple Court, Atlanta, Ga. Born, Rock Hill, S. C., September 21, 1875. Education, two years Wofford College, also common school. Course in electric lighting and railway work. For four years assistant engineer Tallassee Falls Manufacturing Company, Tallassee, Ala. May 1, 1903, appointed engineering aid in the reclamation service; January 1, 1904, promoted to assistant engineer.

GORDON, CHARLES EDWARD, assistant engineer. Address, Lawton, Okla. Born at Mamaroneck, N. Y., January 20, 1874. Graduated Glens Falls Academy, 1892. Graduated in civil engineering from Union University, 1896. November, 1892–December, 1895, draftsman for irregular intervals, Schenectady Locomotive Works. December, 1895–September, 1896, acting leveler, New York State canals; surveys and estimates. September, 1896–June, 1897, topographer and assistant engineer, making maps and building roads in Adirondack Mountains for W. Seward Webb. November, 1897–June, 1898, with New York State Forest Preserve Board as assistant engineer Indian Lake reservoir. July, 1898–July, 1899, with United States Deep Waterway Commission, water supply division, in charge of stadia topographical party and in charge of borings. July, 1899–December, 1899, assistant engineer on construction, Chicago and Northwestern Railway. December, 1899–December, 1900, topographer, Choctaw,

Oklahoma and Gulf Railroad. December, 1900–August, 1901, resident engineer in charge construction, same railroad. August, 1901–March, 1902, resident engineer in charge construction, Choctaw, Oklahoma and Texas Railway. March, 1902–December, 1902, resident engineer, in charge of construction, Rock Island System. December, 1902–June, 1903, resident engineer, in charge of construction, Missouri, Kansas and Texas Railway. June 1, 1903, appointed assistant engineer in the reclamation service.

GRANKE, LEO ERNEST, assistant engineer. Address, Washington, D. C. Born in Wisconsin, October 2, 1877. Graduate of La Crosse (Wis.), high school; Wisconsin Business College; University of Wisconsin, B. C E., 1900. 1898–1900, rodman, leveler man, and field assistant, United States Geological Survey. In 1900 was with Chicago and Alton Railroad. 1901–2, assistant engineer, Chicago and Alton Railroad. 1902–3, assistant topographer, United States Geological Survey. September 1, 1903, appointed assistant engineer in the reclamation service.

GREEN, HUBERT EDWARD, engineer. Address, San Francisco, Cal. Born in England, August 17, 1859. Education, Auckland, New Zealand, college and grammar school. Partial course in electrical engineering with International Correspondence School. Member American Society of Civil Engineers. From 1874 to 1884 was with public works department, New Zealand Government. 1884–1886, construction engineer Sierra Valley and Mohawk Railway. 1886–1888, in city engineer's office, San Diego, Cal. 1888–1890, with International Company of Mexico; also made reconnaissance survey for railroad. May–December, 1890, in charge of construction of railroad trestle across Elliott Bay in Puget Sound; also designed and put in grillage foundations and superintended erection of swing bridge across White River. 1891, in charge of location in the field, canal survey, Northern Pacific Railroad. 1891–1892, locating and construction engineer for Texas, Louisiana and Eastern Railroad; also, in 1893, in charge of levels for Colorado River Irrigation Company. 1893–94, consulting engineer in San Diego City and County. 1894–1896, chief engineer Coronado Beach Company; surveys of reservoirs for San Diego water supply; afterwards superintended laying of foundation of Morena Dam. 1897, examined and report on placer mine in Cariboo, British Columbia. 1897–98, general practice in San Francisco. 1899, in charge of construction and draftsman in office Los Angeles electric railroads. 1900–1902, field assistant, United States Geological Survey. 1902–3, in charge of surveys, etc., for Seaside Water Company, Long Beach, Cal.; also subdividing tract of land for Beach Land Company, of Los Angeles, Cal.; made plans and estimates for water and sewer systems and improvements for seaside resort at Playa del Rey. May 4, 1903, appointed engineer in the reclamation service.

GROVER, NATHAN CLIFFORD, engineer. Address, Orono, Me. Born, Bethel, Me., January 31, 1868. Education, University of Maine, B. C. E. and C. E.; Massachusetts Institute of Technology, B. S. 1889, location and construction, Somerset Railway. 1890, location and construction of extension, Maine Central Railroad. 1891–1894, assistant in civil engineering, Maine State College. 1894, water-power department, Rumford Falls, Me. 1895, leveler on construction, Sebasticook and Moosehead Railroad. 1894–1896, assistant professor of civil engineering, University of Maine, and associate professor from 1896–97. Professor of civil engineering, University of Maine, 1897–1903. 1901–1903, resident hydrographer, United States Geological Survey. April 29, 1903, appointed hydrographer in the reclamation service. July 1, 1903, designation changed to engineer.

HALL, BENJAMIN MORTIMER, consulting engineer. Address, Temple Court, Atlanta, Ga. Born in South Carolina, January 31, 1853. Education, Emory College, Oxford, Ga., and University of Georgia; B. E. and C. E. and M. E. 1876–1880, professor of mathematics, Northern Georgia Agricultural College, Dahlonega, Ga. 1880–1884, chief engineer in water-supply investigations, surveys, and construction of mining canals, flumes, tunnels, etc., for hydraulic mining. 1884–1887, superintendent of

Gold Mines Hydraulic Mining, Lincoln County, Ga., and underground mining in Warren County. 1887–1890, superintendent Southern Marble Company, Marble Hill, Ga. 1890–1903, in general civil and hydraulic engineering practice, Atlanta, Ga. 1895–1903, resident hydrographer, United States Geological Survey. Has made water-supply investigations for Atlanta, Columbus, Dublin, and Elberton, Ga. 1903, consulting engineer, United States Geological Survey. Author of "Water Powers of Georgia, 1896;" "Water Powers of Alabama, 1903;" in preparation "Water Powers of Georgia, 1903."

HALL, MAXCY REDDICK, hydrographer. Address, Temple Court, Atlanta, Ga. Born, Georgia, November 16, 1864. Education, common school and University of Georgia. 1895–1903, assistant resident hydrogapher, United States Geological Survey. In 1900 made water-power surveys on Saluda River, near Newberry, S. C., and Broad River, at Anthony Shoals, Ga.; 1901, on Chattahoochee River, near Buford, Ga., and on Yellow River, near Conyers, Ga.; 1902, on Hiawassee River, near Murphy, N. C. 1889–1903, general civil and hydraulic engineering, Atlanta, Ga. January 1, 1903, appointed hydrographer in the reclamation service.

HALL, WARREN ESTERLY, engineering aid. Address, Temple Court, Atlanta, Ga. Born in Georgia, October 5, 1881. Education, Georgia School of Technology, C. E. course. 1899, hydrographer, United States Geological Survey; also ran levels for preliminary survey of Gainesville railroad; also surveying for Consolidated Mining Company, Dahlonega, Ga. 1899–1900, levelman on navigation survey in Chattahoochee Corps of United States Engineers. 1900, assistant hydrographer in Appalachian Park investigation. 1901–2, assistant engineer on water-power survey for Chattahoochee Falls Company, Columbus, Ga.; also assistant engineer on Garner water-power survey, on upper Chattahoochee River. 1902–3, assistant hydrographer for United States Geological Survey. 1903, on hydrographic and topographic surveys for Hall Brothers, Atlanta, Ga. January 6, 1903, appointed engineering aid in the reclamation service.

HAMLIN, HOMER, engineer. Address, Los Angeles, Cal. Born at Pine Island, Minn., August 27, 1864. Education, common school and high school, and independent study. 1887–1893, was with city engineer, San Diego, Cal. 1893–1895, general engineering and surveying business at San Diego, Cal. In 1894 was assistant to United States engineers in survey for fortifications on Point Loma and Coronado Beach. 1895, draftsman, Los Angeles, Cal.; also in surveyor's office, of Los Angeles County. 1895–96, in office of city engineer, Los Angeles, Cal. 1897–98, in office of county surveyor of Los Angeles County. 1899–1900, deputy city engineer, Los Angeles, Cal. 1901, hydrographic and geologic investigations for United States Geological Survey. 1902, hydrographic and hydrologic investigations for United States Geological Survey. January 1, 1903, appointed engineer in the reclamation service.

HANNA, FRANK WILLARD, assistant engineer. Address, 263 Dearborn street, Chicago, Ill. Born in Geneseo, Ill., September 16, 1867. Education, Western Normal College, Shenandoah, Iowa, and Highland Park College, Des Moines, Iowa. 1894–95, was assistant in departments of mathematics and civil engineering in Highland Park College, and in 1895 became dean of departments named, continuing as such until 1903. In 1900, commissioned by governor of Iowa to investigate foreign schools and traveled through Europe studying engineering structures. Is author of pamphlet "Logical methods in arithmetic," and inventor of an "angle multisector." June 1, 1903, appointed assistant engineer in the reclamation service.

HANSEN, ANDREW C., engineering aid. Address, Los Angeles, Cal. Born, Denmark, February 18, 1880. Education, University of California. In 1902 was chainman, rodman, and computer for N. B. Kellogg, San Francisco, in locating power plant on Feather River, also ran compass and took slopes. May 15, 1903, appointed engineering aid in the reclamation service.

HARDESTY, WILLIAM PRESTON, assistant engineer. Address P. O. box "S," Salt Lake City, Utah. Born in Missouri April 5, 1865. Education, college and university, University of Missouri, C. E. 1887–88, with Interstate Consolidated Rapid Transit Railway Company, Kansas City, Mo. 1888, with Denver Tramway Company, Denver, Colo. 1889, with Denver City Railway Company. 1890, with Denver and Rio Grande Railway, assistant engineer on construction, Colorado canal. 1891, assistant engineer for T. C. Henry Canal Companies, San Luis Valley, Colo. 1892, assistant engineer for Lemhi Placer Gold Mining Company, Idaho. 1893–1903, private engineering practice, Salt Lake City, Utah. June 5, 1903, appointed assistant engineer in the reclamation service.

HARLEY, GEORGE FOSTER, engineering aid. Address, Sparta, Ga. Born in Georgia January 6, 1875. Education, high school, university, and scientific school. In 1898 was employed on preliminary railroad survey in Georgia, and in same year received civil service appointment in Department of Agriculture in East St. Louis, Ill.; in 1900 transferred to Department of Interior at Washington, D. C. July 1, 1903, appointed engineering aid in the reclamation service and assigned to work in State of Washington.

HARRIS, ARCHIE LEE, assistant engineer. Address, Livingstone, Ariz. Born in Orange, Mass., February 17, 1872. Education, University of Michigan, B. S. in C. E. 1894–95, teaching school. 1898–1900, rodman and draftsman on United States deep waterways survey along Hudson River, and in office at Detroit, Mich. 1900–1901, draftsman and computer on New York State Barge Canal survey, Albany, N. Y. 1901, draftsman for United States Isthmian Canal Commission, Washington, D. C. 1901–2, draftsman for Russell Wheel and Foundry Company, Detroit, Mich., and Hamilton Bridge Works Company, Hamilton, Ontario. 1902–3, draftsman for Pennsylvania, New York and Long Island Railroad Company, New York. May 19, 1903, appointed assistant engineer in the reclamation service.

HAWLEY, RALPH STEVENSON, assistant engineer. Address, Los Angeles, Cal. Born in California, January 11, 1877. Education, high school and University of California, electrical and civil engineering courses, B. S. in C. E. In 1902, in city engineer's office, Oakland, Cal., and with Spreckels Sugar Company. In 1903, assistant in University of California Summer School of Civil Engineering, at Monterey, Cal. During 1902–3 was twice chosen president of civil engineering association of University of California. August 7, 1903, appointed engineering aid in the reclamation service; January 1, 1904, promoted to assistant engineer.

HAWLEY, ROBINSON WILBER, assistant engineer. Born in New York, September 28, 1876. Graduate of Colorado Agricultural College in irrigation engineering. 1896–97, assistant engineer, irrigation surveys on the Rio Puerco and canal superintendent in Idaho. 1898, assistant engineer on survey for canal in Rio Grande Valley. 1899–1900, designing irrigation structures and drafting and on stream measurements for State in Colorado. 1901, topographic surveys for lateral systems. 1901–2, with Oregon Short Line; also with Mammoth Reservoir Company, Fairview, Utah; also, same year, on surveys, United States Geological Survey, Montrose, Colo.; January 1, 1903, appointed assistant engineer in reclamation service.

HAYDEN, BEAUFORD EMMETT, assistant engineer. Born in Missouri, August 23, 1875. Education, high school, private college, and Territorial University, Norman, Okla.; B. S. in 1900. In 1901 and 1902, constructed ditches and laterals for the reclamation of land near Yuma, Ariz. In 1902–3, on topographic work on Colorado River. June 9, 1903, appointed engineering aid in the reclamation service, and assigned to duty in the Salt River Valley, Arizona; December 9, 1903, promoted to assistant engineer.

HAYS, DAVID WALKER, assistant engineer. Address, Reno, Nev. Born in Bridgeport, Cal., March 24, 1878. Education, Nevada State University, B. S. In 1900, was levelman and topographer under commissioner of State highways of the

State of California, in survey of road into Yosemite Valley, and in survey of Placerville road between Placerville and Lake Tahoe. 1901–2, assistant hydrographer and engineer in stream measurements and surveys of canals and reservoirs, under L. H. Taylor, of United States Geological Survey. January 1, 1903, appointed assistant engineer in the reclamation service.

HENDRICKS, ERNEST DEMAREST, engineering aid. Address, Las Cruces, N. Mex. Born in New York, December 16, 1879. Education, Cornell University, C. E. During summers 1897–1900 with Hendricks Haypress Manufactory, Kingston, N. Y. In 1901, rodman with geological survey of New York State. In 1902, in New York State engineer corps. June 13, 1903, appointed engineering aid in the reclamation service.

HEWITT, CHARLES EDGAR, engineering aid. Address, Peyton Block, Spokane, Wash. Born in Iowa, October 30, 1868. Education, Iowa City Academy, commercial school, and State University of Iowa, C. E. During 1897 and 1898 was with United States Geological Survey as levelman, then accepted position on engineer corps of Chicago, Burlington and Quincy Railway; during field seasons since that time has been with Government survey. 1901–2 with Missouri, Kansas and Texas Railroad on location work in Oklahoma and Indian Territory. July 16, 1903, appointed engineering aid in the reclamation service.

HILL, LOUIS CLARENCE, engineer. Address, Livingstone, Ariz. Born in Michigan, February 22, 1865. Education, University of Michigan, B. S. in C. E. and E. E. In 1886–87 was draftsman with Detroit Pipe and Foundry Company, Detroit, Mich. 1887, leveler and transitman, Duluth, Redwing and Southern Railroad, also in St. Paul office of United States Engineer Corps as assistant engineer. 1887–88, transitman and chief of party, Duluth, Redwing and Southern Railroad. 1888, resident engineer Great Northern Railroad, St. Paul, Minn. 1890–1903, professor of physics and electrical engineering, Colorado School of Mines. In 1893 served as judge on committee of awards, World's Columbian Exposition, and since that date has served many times as consulting engineer. June 8, 1903, appointed engineer in the reclamation service.

HINDERLIDER, MICHAEL C., engineer. Address, Chamber of Commerce building, Denver, Colo. Born, Medora, Ind., May 19, 1876. Education, public schools and University of Techology of Indiana, B. S. in C. E., Purdue University. In 1897 and 1898 was on United States Land Survey in Colorado. During the summers of 1899 and 1900 was employed by the State engineer of Colorado to make stream gagings. 1899–1900, in engineering department of board of public works of Denver, Colo. In 1898 and 1901, in designing and construction, also drainage surveys. In 1902 was engaged in hydrographic work over the State of Colorado under the United States Geologic Survey. January 1, 1903, appointed assistant engineer in the reclamation service; September 1, 1903, promoted to engineer. January 1 to July 31, 1903, in charge of hydrographic work in Colorado under the United States Geological Survey. August 1, 1903, given general supervision of hydrographic work in Colorado and adjoining States, with headquarters at Denver.

HOGUE, GILBERT H., engineering aid. Address, Los Angeles, Cal. Education, University of Idaho, B. S. in C. E. 1900–1901, on maintenance of way and construction work; also, in 1901, assistant engineer and draftsman in office of city engineer, Moscow, Idaho. 1901–2, general engineering and land surveying; also, in 1902, city superintendent on construction, Moscow, Idaho, and chief draftsman and topographer Cascade Canal Company, Ellensburg, Wash. 1902–3, assistant irrigation engineer and hydrographer, United States Geological Survey. January 12, 1903, appointed engineering aid in reclamation service. 1903, topographic work on Minidoka project, Idaho.

HOLGATE, HARRY LANGFORD, clerk. Born, Corvallis, Oreg., January 21, 1867. Graduate of Oregon Agricultural College, B. S. Admitted to practice law in the

courts of Oregon, October 3, 1893. Practiced law in Oregon until departure to the Philippines as member of the Second Oregon Regiment Volunteer Infantry. Appointed clerk United States Census Office in 1900. Assistant chief of irrigation section until June 1, 1903, when he was transferred to United States Geological Survey as clerk in the legal department of the reclamation service.

HOLLISTER, GEORGE BUELL, hydrographer. Address, Washington, D. C. Born, Brooklyn, N. Y., September 16, 1865. B. A., Yale University. From 1892 to 1896 was private secretary to Arthur M. Dodge, New York City, and during that time was also assistant treasurer of Central Electric Heating Company, and treasurer of Dewey Elecrtric Signal Company. 1896–97, with estate of Arthur M. Dodge; then entered employ of his father in wholesale grain and feed business, New York City. 1901–1903, hydrographer, United States Geological Survey. January 1, 1903, appointed hydrographer in the reclamation service.

HOLT, LESTER MORTON, assistant engineer. Address, Lawton, Okla. Born, Wisconsin, February 25, 1877. Education, Columbian University; degree B. S. in 1900; degree of C. E. in 1902. Member of engineer corps of District of Columbia National Guard for four years, making maps, etc. From 1896 to 1902 was messenger, clerk, and bookkeeper, United States Treasury Department. July 20, 1902, appointed irrigation assistant in United States Geological Survey. July 1, 1903, appointed assistant engineer in the reclamation service. Four months' instrument work on surveys of Grand Valley irrigation project, Colorado. One month's office work, level and transit, at Montrose, Colo. Three months' plane-table and level at Mountain Park, Oklahoma Territory. Four months' office work, level, plane-table work, etc., for investigation for Lawton waterworks, Lawton, Okla.

HORTON, ALBERT HOWARD, engineer. Address, Silver Creek, N. Y. Graduate of Cornell University, degree C. E. In 1898–99 was stadiaman and draftsman on United States Deep Waterway Survey; also, in 1899, rodman in precise-level party. 1899–1900, recorder with United States Lake Survey, engaged in hydrographic surveys and discharge measurements on the St. Clair River. 1900–1901, recorder with United States Lake Survey, in charge of field work in connection with discharge measurements of St. Lawrence River. 1901–2, with United States Lake Survey, in charge of office work connected with a resurvey of St. Lawrence River. 1902–3, junior engineer with United States Lake Survey, in charge of precise level party, running levels between Greenbush and Oswego. May 9, 1903, appointed assistant engineer in the reclamation service; January 1, 1904, promoted to engineer.

HORTON, ROBERT ELMER, hydrographer. Address, 75 Arcade, Utica, N. Y. Born in Michigan, May 18, 1875. Education, Albion College, Michigan, degree B. S. 1895–1897, made calculations of rainfall and run-off of Hudson River for New York State engineer. In 1898 was second assistant engineer of construction, Indian Lake storage dam, New York. 1898–99, computer, water-supply division, United States Deep Waterways Survey; also, same year, city paving inspector, Rome, N. Y. 1899–1900 and later, in private practice. Investigated water powers on West Canada Creek, New York, built storage pond, Oneida County; made surveys and plans. Fishkill (N. Y.) waterworks, and Binghamton (N. Y.) water powers; consulting expert in various cases, etc. 1900–1902, hydrographer, United States Geological Survey, in charge of stream gagings in New York State, and later in Michigan. January 1, 1903, appointed hydrographer in the reclamation service.

HOSFORD, CLARENCE KENT, engineering aid. Born in Vermont, September 17, 1875. Education, Dartmouth College, B. S., and Thayer School of Civil Engineering. In 1900 assisted in contour work and laying out foundations. In 1902, field assistant with United States Geological Survey. January 5, 1903, appointed engineering aid in the reclamation service and assigned to duty in New Mexico.

HOYT, JOHN CLAYTON, computer. Address, Washington, D. C. Born at Lafayette, N. Y., June 10, 1874. Education, Andover Academy and Cornell University; gradu-

ated in civil engineering in 1897. From May to October, 1897, was with Cornell University hydraulic laboratory construction, as assistant engineer. From November, 1897, to October, 1898, was with United States Deep Waterways Commission on Oswego route. November, 1898, to September, 1899, draftsman, Bureau Yards and Docks, Navy Department, Washington, D. C. October, 1899, to August, 1902, with United States Coast and Geodetic Survey, as computer. September 3, 1902, appointed computer in United States Geological Survey.

HUBER, FREDERICK W., engineering aid. Address, Reno, Nev. Born in Virginia, August 31, 1879. Education, Baltimore City College and Cornell University. In 1899, was rodman on preliminary surveys for United Railways of Baltimore; in 1900, rodman on construction, United Railways of Baltimore; in 1901, rodman on location, Baltimore and Ohio Railroad; 1902, temporary assistant engineer, United Railways of Baltimore. May 7, 1903, appointed engineering aid in the reclamation service.

HUFFAKER, ISAAC WATTS, engineering aid. Address, Reno, Nev. Born in California, September 25, 1874. Education, common school and private instruction in surveying, etc. 1899 to 1901 was in general-surveying office at Deer Lodge, Mont. In 1901 was employed in construction of iron-frame smelters at Anaconda, Mont. May 7, 1903, appointed engineering aid in the reclamation service. Employed as hydrographer on Walker River Basin for three months following appointment; since that time as levelman on Lower Truckee canal.

HUMPHERYS, THOMAS HYRUM, engineering aid. Address, 391 South Main street, Logan, Utah. Born in Idaho, November 26, 1874. Graduate of Utah Agricultural College, degree B. S. in C. E. In 1897–98 was employed at Utah Experiment Station, United States Department of Agriculture. In 1898, surveying for Newton Reservoir Company, Newton, Utah. In 1900, on work connected with subdivision of Government lands. 1901–2, surveyor for Cache County, Utah. 1902–3, employed by the West Cache Canal Company, of Trenton, Utah. June 15, 1903, appointed engineering aid in the reclamation service.

HUMPHREYS, DAVID CARLISLE, assistant hydrographer. Address, Lexington, Va. Born in Virginia, October 14, 1855. Education, Washington and Lee University, degree C. E. 1872–1874, engaged in mapping and general surveying. 1874–75, office assistant and draftsman, railway construction. 1878–79, teacher. 1879–1885, draftsman and United States assistant engineer on improvement of Mississippi and Missouri rivers, under board of engineers and Missouri River Commission. 1885–1903, professor of civil engineering at Washington and Lee University. In 1888 was engineer for increasing water supply of Lexington, Va. Also, at various times, engineer and architect for land companies, railroads, etc.; also consulting engineer and vice-president, Lexington Light and Power Company. 1895–1903, resident hydrographer, United States Geological Survey. February 2, 1903, appointed hydrographer in reclamation service.

HURD, HUED CLARENCE, assistant engineer. Address, Roswell, N. Mex. Born in Texas, April 16, 1870. Graduate of Princeton, C. E. During 1891 spent four months on the construction of the Broadway cable road, New York City. 1893–1897, degree with engineering department of District of Columbia. With Nicaragua Canal Commission, 1897–1899, and with Isthmian Canal Commission, 1899–1901; then accepted position as transitman with Washington and Annapolis Electric Railway Company. In 1902, assistant engineer with J. M. Guffey Petroleum Company, Port Arthur, Tex. September–December, 1902, with special agent, Department of State, on work on Nicaragua Canal route. May 1, 1903, appointed assistant engineer in the reclamation service.

JOHNSON, EDWARD, Jr., engineer. Address, 263 Dearborn street, Chicago, Ill. Born in Boston, Mass., December 4, 1876. Education, Massachusetts Institute of Technology, degree B. S. September 26, 1899, to October 13, 1900, with Essex County, Lawrence, Mass., as assistant engineer in construction work, water measure-

ments, hydraulic engineering, and city surveying. October 25, 1900, to December 15, 1902, attached to U. S. S. *Ranger* as hydrographic surveyor. Winter of 1900–1901, connected with building of wharf for United States coaling station at Pichilinque Bay, Mexico, and during that time and in 1902 executed the main triangulation of the survey of the coast of Lower California. February 2, 1903, appointed assistant engineer in the reclamation service; January 1, 1904, promoted to engineer.

KEDDIE, WILLIAM ARTHUR, assistant engineer. Address, Reno, Nev. Born in California March 4, 1878. Education, Nevada State University; degree B. S. In 1900, on topographical surveys; also made topographic maps and did general drafting for report on Truckee River project. 1901–2, draftsman in United States surveyor-general's office. June–September, 1902, on United States survey work in Nevada. 1902–3, made preliminary surveys of canal lines along Truckee River. January 24, 1903, appointed assistant engineer in the reclamation service, since which time has been on topographic surveys in Nevada.

KERR, JOHN NICEWAUNGER, assistant engineer. Address, Monte Vista, Colo. Born in Ohio, July 1, 1879. Education, University of Colorado and Colorado School of Mines. In 1898 was field assistant with Colorado Valley Land and Canal Company. 1899, assistant engineer with Colorado Valley Land and Canal Company. 1900, mining at Summitville and Ouray, Colo. 1902, draftsman with Southern Colorado Land Company; also mining and mining surveying, Summitville district, Colorado. September 15, 1902, appointed assistant engineer in the reclamation service.

KLEINSCHMIDT, HENRY SCHWING, engineering aid. Address, 2112 Waverly place, St. Louis, Mo. Born in Tennessee, May 28, 1882. Graduate of University of Missouri, degree B. S. in C. E. In 1901 was with survey party in Oklahoma and Texas, as chainman for St. Louis and San Francisco Railroad. In 1902, with Atchison, Topeka and Santa Fe Railroad, between Chicago and Kansas City, on maintenance of way. June 11, 1903, appointed engineering aid in the reclamation service.

LAURGAARD, OLAF, engineering aid. Address, La Crosse, Wis. Born in Norway, February 21, 1880. Education, high school and University of Wisconsin. In 1901 had charge of level party in northern Minnesota for United States Steel Corporation. 1902, running levels for electric railway. 1902, with United States Geological Survey as primary levelman and traverseman. May 14, 1903, appointed engineering aid in the reclamation service.

LAWSON, LAWRENCE MILTON, assistant engineer. Address, Los Angeles, Cal. Born Washington, D. C., January 8, 1879. Education, high school, Polytechnic Institute of Pasadena, Cal., Mount Tamalpais Military Academy, and Stanford University. In 1902 was in charge of pipe-line survey for Pomona and Ontario Gas and Fuel Company, Los Angeles, Cal.; also in charge of party making stream measurements in California for United States Geological Survey. January 5, 1903, appointed engineering aid in the reclamation service; July 5, 1903, promoted to assistant engineer.

LEIGHTON, MARSHALL ORA, hydrographer. Address, Washington, D. C. Born in Maine, May 1, 1874. Graduate of Massachusetts Institute of Technology. From 1896 to 1902 was executive officer for the board of health at Montclair, N. J., and during the latter half of this period was engaged in private sanitary-engineering practice as consulting expert. In 1899 was associated with George W. Fuller, consulting expert, New York City, in an examination of drainage area of Passaic River above Little Falls to determine the fitness of the water flowing therefrom for use in domestic supply. In 1901 was employed by the United States Geological Survey to prepare a report on sewage pollution in rivers around New York, published as Water Supply and Irrigation Paper No. 72. In 1902 was engaged in a new branch of work under the United States Geological Survey having especial reference to pollution of waters and the damage done thereby, and reviewed water investigations made in the northeastern part of the United States in a report published as Water Supply and Irrigation Paper No. 79. Also investigated, in conjunction with Mr. George B. Hollister,

the great floods on the Passaic River in March, 1902, and October, 1903. Made an extended trip through Indiana, Illinois, Iowa, Missouri, and Arkansas, exploring river drainage areas for locations of current-meter measurement stations, and on December 1, 1902, was put in charge of river measurements in the Mississippi district, which covers the above-named States, together with Kentucky, Ohio, Wisconsin, and Minnesota. January 1, 1903, was appointed hydrographer, and put in charge of the division of hydro-economics.

LEMÉNAGER, HENRI V., draftsman. Address, Washington, D. C. Born in England, 1870. Education, English high school and special courses in art in South Kensington School of Science and Arts. 1889–1894, draftsman with Northern Pacific Railroad; 1894–95, draftsman, United States Surveyor-General's Office; 1900, draftsman, Rand-McNally Company. In January, 1901, was appointed draftsman, United States Geological Survey, and on October 1, 1902, assigned to duty in the hydrographic branch.

LEWIS, JOHN HOWARD, hydrographic aid. Address, Pendleton, Oreg. Born in Oregon, April 22, 1878. Education, graduate of high school, three years in Stanford University, C. E., Cornell, 1903. In 1899 and 1900 was draftsman for Oregon Railroad and Navigation Company; also, in latter year, chief topographer in field for same company, and from August, 1900 to September, 1901, was in charge of records of right of way department of same company; June to September, 1902, was field draftsman on construction work for same company. June 10, 1903, appointed hydrographic aid in the reclamation service, having charge of all hydrographic work in the State of Oregon.

LIPPINCOTT, JOSEPH BARLOW, supervising engineer. Address, Los Angeles, Cal. Born in Scranton, Pa., October 10, 1864. Educated at Dickinson College, Pennsylvania, and University of Kansas, receiving degree of B. S. in 1886. Since that time has been continuously engaged in civil engineering. 1886–1888, railroad work for the Santa Fe System. In July, 1888, was employed by United States Geological Survey for topographic work and continued until 1892, when he was appointed assistant engineer for Bear Valley Irrigation Company, of Redlands, Cal., and assisted in building a portion of the Santa Ana canal. In 1893 took charge of field work of an enterprise contemplating irrigation of 30,000 acres in northwestern portion of Los Angeles County, Cal.; work abandoned in December, 1894. In January, 1895, was placed in charge of hydrographic investigations for the United States Geological Survey in California and has had charge of said work to date. Is author of various United States Geological Survey publications. Has been employed on domestic waterworks of various cities, including San Francisco, Los Angeles, and Santa Barbara, Cal., as consulting engineer and also in same capacity by a number of irrigation companies.

LOORZ, ALF C., assistant engineer. Address, Reno, Nev. Born in Massachusetts, March 8, 1870. Graduate of Massachusetts Institute of Technology; degree, S. B.; also graduate of English High School, Boston. In 1896–97 rodman with Boston Transit Commission. 1897–98, with chief engineer city of Boston, when he had charge of field work, laying out and estimating and making surveys. 1898–1903, in Navy Department at Boston Navy-Yard, in surveying, inspecting construction of quay walls, buildings, etc.; also had charge of laying out the new dry dock at that yard, making estimates for same. June 4, 1903, appointed assistant engineer in the reclamation service.

MCCONNELL, IRA WELCH, engineer. Address, Montrose, Colo. Born in Missouri, October 17, 1871. Graduate of Cornell; degree, C. E. In 1897, contractor's foreman; 1898, topographic surveyor, United States Deep Waterways Commission; same year, rodman, United States navy-yard, Brooklyn, N. Y. 1898–99, contractor, North Tarrytwon, N. Y., sewerage system; 1899–1900, instructor in civil engineering, Cornell University; 1900–1901, superintendent of construction, Indian-

apolis Water Company; 1901–1903, superintendent, Nash & Dowdle, contractors, Chicago and New Orleans; January to June, 1903, professor of civil engineering, Missouri School of Mines; June 14, 1903, appointed hydrographer in the reclamation service; December 14, 1903, designation changed to engineer.

McDERMITH, ORO, engineering aid. Address, Chamber of Commerce Building, Denver, Colo. Born in Colorado, August 24, 1879. Graduate of Colorado Agricultural College; degree, B. S. In 1894–95 was miner and prospector, Leadville and Gunnison, Colo. 1895–1897, ranchman and practical irrigator, Eagle, Colo. 1897–1899, student at college, Fort Collins, Colo. 1899–1900, clerk and telegraph operator, Wolcott, Colo. 1900–1902, student, Fort Collins, Colo. 1902–3, fellow in irrigation engineering, and assistant in irrigation investigations at Agricultural Experiment Station, Fort Collins. May 15, 1903, appointed engineering aid in the reclamation service.

McINTYRE, OTIS LITTLE, engineering aid. Address, Washington, D. C. Graduate of Columbian University. 1901–1903, work in general engineering, cartographic draftsman in United States hydrographic office. May 18, 1903, appointed engineering aid in the reclamation service.

MACPHERSON, DAVID J., assistant engineer. Address, Pasadena, Cal. Born in Canada, January 12, 1854. Graduate of Cornell University, B. C. E. 1878–1880, with Worthington and Sioux Falls Railroad in Minnesota. 1881–82, building railroad projects under contract. 1883, assistant city engineer, San Antonio, Tex. 1884, locating engineer, Mexican National Railway. 1885–1891, in real estate business in California. 1892–1896, chief engineer Mount Lowe Railway. 1899–1901, chief engineer, constructing railway and installing mining machinery for parties in Mexico. 1902–3, on hydrographic work, United States Geological Survey. December 5, 1903, appointed assistant engineer in reclamation service.

MADDEN, FRANCIS MICHAEL, hydrographic aid. Address, Chamber of Commerce Building, Denver, Colo. Born in Minnesota, October 10, 1878. Graduate of University of Minnesota, C. E. April to September, 1901, was rodman with "Soo" Line. May–September, 1902, instrument man, Chicago Great Western Railway. July 27, 1903, appointed hydrographic aid in the reclamation service.

MAGRUDER, FRANK C., engineering aid. Address, Chamber of Commerce Building, Denver, Colo. Born in Missouri, January 16, 1879. Graduate of University of Missouri; degree B. S. in C. E. In 1901 was topographer on Missouri Pacific Railway. In 1902 was rodman with Missouri, Kansas and Texas Railway. In 1903, instructor in surveying at Missouri State University. June 17, 1903, appointed engineering aid in the reclamation service.

MATTHES, GERARD HENDRIK, engineer. Address, Lawton, Okla. Born in Holland, March 16, 1874. Education, Massachusetts Institute of Technology; B. S. 1895. During summer of 1893 in plan department of Associated Factory Mutual Insurance Companies. 1895–1897, in town engineer's office, Brookline, Mass., as draftsman, instrument man, and sewer inspector; also instrument man for three months for Massachusetts State Board of Harbor and Land Commissioners. Since May, 1897, to date with United States Geological Survey, on stream gaging, reservoir and canal surveys. At present district engineer in charge of reclamation surveys and construction of public works in Oklahoma, and special disbursing agent.

MEEKER, RALPH INMAN, engineering aid. Address, Chamber of Commerce Building, Denver, Colo. Born in Kansas, February 14, 1878. Education, high school and Colorado State School of Mines. In 1899–1900 was assistant to the chief engineer of the Twin Lakes Land and Water Company and Twin Lakes Reservoir Company. In 1900–1901 was with Union Gold Extraction Company, Florence, Colo., as head barrel man and precipitator. July–August, 1901, with The Woods Investment Company, at Economic Chlorination mill, Cripplecreek, as barrelman. In 1902 assayer in office of A. E. Reynolds, Pueblo, Colo.; also, in 1902, in mine surveying and sam-

pling at Blackhawk, Colo., with Boston and Denver Consolidated Mining and Milling Company; also, same year, with Colorado Northern Gold Mining Company, as assayer and electrician, also mine surveying. In 1902–3, with Wall Street Gold Extraction Company, Wallstreet, Colo., as general man. May 1, 1903, appointed engineering aid in reclamation service.

MILLER, GEORGE S., engineer. Born at Lowell, Mass., April 22, 1872. B. S. in C. E., University of Vermont. 1895–96, assistant engineer, United States engineers' office, Burlington, Vt.; also, in 1896, with Boston and Albany Railroad, on separation of grades. 1896–97, with Metropolitan Sewerage Commission, at Dedham, Mass. 1897–98, with Murphy & Haskins, contractors. 1898–1904, with United States Engineers, War Department, as assistant superintendent and superintendent. March 9, 1904, appointed engineer in the reclamation service, by transfer from War Department.

MORRIS, ANTHONY PAUL, hydrographic draftsman. Address, Washington, D. C. Born in Maryland, March 21, 1876. Education, high school and normal school; now attending Columbian University, Washington, D. C. In 1894 was employed as assistant in making subdivision surveys in District of Columbia. 1895–1897, with United States Fish Commission. 1898–99, served in United States Army during Spanish-American war. In February, 1901, was appointed hydrographic draftsman in United States Geological Survey.

MURPHY, EDWARD CHARLES, engineer. Address, Washington, D. C. Born in Canada, June 17, 1859. Graduated from Cornell University in 1884; degree, B. C. E. Has since received from that university the degrees of M. S. and Ph. D. Fellow in mathematics, Cornell, 1885–86. 1887–1898 was assistant professor of civil engineering at University of Kansas. In 1899 fellow in civil engineering, Cornell University. In 1900 hydrographer, United States Geological Survey. In 1901 instructor in civil engineering at Cornell. 1892–1898, resident hydrographer, United States Geological Survey, for eastern Kansas. 1902, hydrographer and assistant engineer, United States Geological Survey. Associate member American Society of Civil Engineers. Has published Water-Supply and Irrigation Papers Nos. 8, 42, 43, and 64, also papers in Kansas University Quarterly, Kansas Academy of Science, Engineering News, and Transactions American Society of Civil Engineering. October 9, 1902, appointed assistant engineer in the reclamation service. July 1, 1903, promoted to engineer.

MYERS, EDWARD WARREN, assistant engineer. Address, Greensboro, N. C. Born in North Carolina, October 30, 1873. Education, private school, graded school, military preparatory school, and university. Graduated with degree B. S. from University of North Carolina in 1895. Associate member American Society of Civil Engineers. Since his graduation has been employed for the greater part of the time as engineer of the North Carolina geological survey in reconnaissances and investigations of water power of North Carolina, topographic surveys, etc.; also as resident hydrographer for North and South Carolina, in charge of the hydrographic work in these States and in parts of Virginia and East Tennessee. Private work has consisted of surveys and plans for water-power development, water supply, sewerage, and electric railways, and of expert testimony before courts in suits relative to these subjects. January 1, 1903, appointed assistant engineer in the reclamation service.

NEWELL, FREDERICK HAYNES, chief engineer. Address, Washington, D. C. Born in Bradford, Pa., March 5, 1862. Graduate of Massachusetts Institute of Technology as engineer, 1885. Engaged in mining in Colorado. Miscellaneous engineering in Ohio, Pennsylvania, Virginia, and other States. With United States Geological Survey, 1889–90, as assistant hydraulic engineer; 1890–1895, topographer; 1895–1902, hydrographer. July 8, 1902, appointed chief engineer, reclamation service. Author of Hydrography of the Arid Regions, 1891; Agriculture by Irrigation, 1894; The Public Lands of the United States, 1895; Irrigation, 1902. Member commission on revision of land laws.

NEWELL, HERBERT D., assistant engineer. Address, Washington, D. C. Born in Massachusetts, August 6, 1871. Education, common school, academy, and Massachusetts Institute of Technology; degree, B. S. June–September, 1896, surveyor, Erie Railroad. In 1896–97, with Boston Transit Commission. 1897–1902, with United States Engineers, Boston. 1897–98, on survey of Broad Sound, Boston Harbor, and survey of upper Merrimac River. 1898–1902, on fortification construction. 1902–3, at United States engineer's office at Fort Stevens, Oreg., assistant to engineer in charge of fortification construction and in charge of improvement of mouth of Columbia River. June 1, 1903, appointed assistant engineer in the reclamation service.

NEWELL, ROBERT J., engineering aid. Address, Boise, Idaho. Education, common and high school and college. In 1900 taught school. 1901–1903, student at Highland Park College, Des Moines, Iowa. May 14, 1903, appointed engineering aid in reclamation service and assigned to duty as transitman on preliminary work in Idaho on Minidoka project, under J. T. Burke, and later on Dubois proposition and several reservoirs, under J. G. Camp.

NOBLE, THERON AUGUSTUS, engineer. Address, 427 Peyton Block, Spokane, Wash. Born New Philadelphia, Ohio, January 7, 1864. Education, Iowa State College and Massachusetts Institute of Technology. Engaged in civil engineering 1890–1892 for Kirkland Land and Improvement Company, Seattle, Wash. 1892–93, superintendent of machine shop and foundry, Seattle Iron and Steel Manufacturing Company, Seattle, Wash. 1893–1899, civil engineer, general practice, during which time was engaged by Blue Canyon Coal Company; was for three years chief engineer for Seattle Power Company, Seattle, Wash., Pelton Water Wheel Company, San Francisco, Cal., General Electric Company, Portland, Oreg. Member American Society of Civil Engineers. 1899–1901 was civil engineer for Oriental Consolidated Mining Company, Chemulpo, Korea. 1901–1903, engaged in general civil engineering practice and resident hydrographer for the United States Geological Survey. Appointed engineer in the reclamation service May 2, 1903.

OLBERG, CHARLES REAL, assistant engineer. Address, Livingstone, Ariz. Born St. Paul, Minn., August 19, 1875. Graduate Columbian University in civil engineering. Associate member American Society of Civil Engineers. 1894–1896, instrument man on irrigation works, Crow Indian Reservation, Mont. 1896–1900, structural steel draftsman, Supervising Architect's Office, Treasury Department, Washington, D. C. 1900–1902, assistant hydrographer, United States Geological Survey; in charge of field parties on topographic and transit work. 1902 to September, 1903, assistant engineer, reclamation service, in charge of field work on Salt River. Now assistant to L. C. Hill.

OWEN, RAY, engineering aid. Address, 613 Francis Street, Madison, Wis. Born in Wisconsin, October 29, 1878. Education, common school and high school and University of Wisconsin. In 1902 ran levels and did planetable work for United States Geological Survey on East Liverpool and Salineville sheets, in Ohio. June 12, 1903, appointed engineering aid in the reclamation service.

PARSHALL, ADRIAN JEFFERSON, hydrographer. Address, Cheyenne, Wyo. Born in Michigan, May 24, 1850. Grammar and high school education, entering Michigan University in 1867; graduated with degree of C. E. in 1871. In 1871, assistant city engineer at Jackson, Mich.; 1872–1874, draftsman in surveyor-general's office, Cheyenne, Wyo.; 1875, chief clerk in same office, and later, county surveyor of Laramie County, Wyo.; 1876–1880, did private work in the Black Hills, South Dakota; then engaged in banking business. Returned to Cheyenne in 1883 and remained in business until 1895, when he was appointed chief clerk of the State land board, having in charge the work being done under the Carey Act of 1894. 1899–1902, was assistant State engineer of Wyoming, and during that time also performed duties of resident

hydrographer for United States Geological Survey. Since 1902 has given his entire time to the work of the United States Geological Survey. February 2, 1903, appointed hydrographer in the reclamation service.

PATERSON, HARRY THOMAS, assistant engineer. Address, Newport, Ky. Born in Kentucky, September 7, 1874. Education, three years at Rose Polytechnic, Terre Haute, Ind.; six months special work at Missouri School of Mines. October-December, 1895, draftsman with city engineer, Colorado Springs; January-April, 1896, with Wilson & Jackson, contractors on tunnel for Colorado Springs water supply; May-November, 1896, engineer on construction of ditch for Taylor Park Gold Mining Company. In 1897, special work at Missouri School of Mines; 1898–1900, assistant city engineer, Colorado Springs; 1900, assistant superintendent and engineer of Colorado Springs Rapid Transit Railway; 1901–2, deputy county surveyor El Paso County, Colo.; 1902, draftsman, leveler, and transitman on location for Denver and Rio Grande Railway in Colorado. October 28, 1902, appointed assistant engineer in the reclamation service.

PEASE, CHARLES T., engineer. Address, Denver, Colo. Born in Maine, May 3, 1858. Graduate of University of Maine, B. S. University of Denver, LL. B. From 1882–83, assistant engineer in charge of maintenance of way, Mexican National Railway. 1886–87, in charge of residences under construction on Colorado Midland Railway. 1887–88, in charge of divisions under construction, Santa Fe Railroad. 1888–1891, assistant engineer in maintenance of way department, Denver and Rio Grande Railway. 1891, in charge of division under construction on Rio Grande Southern Railway. 1892–1896, in private practice as mining engineer, Rico, Colo. 1897–1901, in United States surveyor-general's office, Denver, Colo. 1901–1903, in private practice as mining engineer. December 14, 1903, appointed engineer in the reclamation service.

PEERY, EDWIN HOWE, law clerk. Address, Washington, D. C. Born in Missouri, July 27, 1856. Graduate of Grand River College, Edinburg, Mo. (A. B.), of Law School, Missouri State University (LL. B.), and of Law School of Columbian University, Washington, D. C. (M. L.). Prior to graduation from college taught school and farmed. 1877–1883, studied law and taught school in California, and was admitted to the bar. 1883, removed to Oregon and clerked in railroad office, land department. 1885–1891, clerk in office of attorneys having railroad and general practice, Portland, Oreg., and assisted in practice. 1891–1894, engaged in law practice, Portland, Oreg. 1894–95, on board of examiners, United States Civil Service Commission. 1895–1897, confidential clerk to Assistant Comptroller of the Treasury. Admitted to the bar of United States Supreme Court in 1897. 1897–1903, law clerk, office of Comptroller of the Treasury. May 12, 1903, law clerk, United States Geological Survey, reclamation service.

PERKINS, EDMUND TAYLOR, engineer. Address, Los Angeles, Cal. Born in Virginia, September 8, 1864. Graduated Union College; C. E. and B. A., 1885; M. A., 1888. From 1885 to date with United States Geological Survey, except parts of 1893 and 1894. In 1893, engineer State Wagon Road Commission of Idaho, equipping and controlling a party which ran reconnaissance lines for wagon roads over 300 miles, through mountainous and unexplored regions. In 1894, United States land inspector of contract surveys; also undertook two contracts for subdividing Government lands. With United States Geological Survey, has been in charge of topographic, triangulation, and leveling parties; 1897–1898, extended triangulation so as to locate by latitude and longitude points on boundary line between Idaho and Montana; also between Idaho and British possessions. July 8, 1902, appointed engineer in the reclamation service and placed in charge of "Yuma project," Colorado River district.

PETERSON, OTTO WALLACE, engineering aid. Born in California, December 31, 1880. Address, 2106 San Pablo avenue, Berkeley, Cal. Education, high school and University of California. In June, 1900, was enumerator for United States census. In 1899–1901 was engaged in farming. In 1902 was rodman for United States drainage survey, near Fresno, Cal. Since September, 1902, has been with United States Geological Survey as hydrographic aid at Los Angeles, Cal., in underground-water investigation and mapping underground waters; also did stream gaging and topographic mapping. May 4, 1903, appointed engineering aid in the reclamation service.

PLACE, ADOLF EDWIN, engineering aid. Address, 53 Sycamore street, Somerville, Mass. Born in Germany, December 21, 1876. Matriculated for one year as student of mechanical engineering and for two years as student of civil engineering at German technical universities of Karlsruhe and Braunschweig. Graduate of Massachusetts Institute of Technology, 1903. Was mechanic, machinist, and draftsman for two and one-half years in Hamburg, Germany. In 1901 was assistant field hydrographer, United States Geological Survey; also civil engineer and surveyor in Boston, Mass. July–September, 1901, employed by city of New York to make investigations of tributaries of Hudson and Housatonic rivers, with reference to future supply of New York City. February–October, 1902, civil engineer with Pierce & Barnes, Boston. June 17, 1903, appointed engineering aid in the reclamation service.

PORTER, ARTHUR PEABODY, engineering aid. Address, Newton, Lower Falls, Mass. Education, public school, including high, Exeter Academy and Massachusetts Institute of Technology. June 6, 1903, appointed engineering aid in reclamation service and served June to August on stream gaging on Long Island and miscellaneous work at Washington, D. C.

PORTER, SAM GRAHAM, engineering aid. Address, 1700 Congress avenue, Austin, Tex. Born in Kyle, Tex., September 1, 1875. Educated at Baylor University (A. B., 1898), University of Chicago and Massachusetts Institute of Technology (B. S., 1903). 1898–1901 taught in Antioch public school, Enon, Tex., East Texas Institute, Rusk, Tex., and Central High School, Waco, Tex. June 10, 1903, appointed engineering aid in reclamation service, United States Geological Survey.

PRALL, CARLOS THEODORE, assistant engineer. Address, 69 Alfred street, Detroit, Mich. Born in Michigan, February 20, 1871. Attended public schools in Detroit and Ypsilanti, Mich., until June, 1886. Began work for hydrographic branch, United States Geological Survey, March, 1899, on Gila River, Arizona. Transferred to Uinta Indian Reservation, Utah, September, 1899; stationed there until August, 1902. Levelman, transitman, and in charge of stream measurements under Cyrus C. Babb. Transferred to Montana August, 1902. In charge of boring party, also plane-table surveys and stream measurements at St. Mary dam site, September–October, 1902. Transferred to Nevada November, 1902. In charge topographic party on Truckee-Carson survey until May 1, 1903. Transferred to Montana May 15, 1903. June–November, 1903, in charge of boring party, also plane-table surveys and stream measurements at St. Mary dam site. June 1, 1903, appointed assistant engineer in reclamation service.

PRESSEY, FRANK ETHELBERT, engineering aid. Address, 487 Union street, Bangor, Me. Born in Maine, April 28, 1879. Education, common school, and graduate of University of Maine, 1902; course in civil engineering. 1902–3 assistant hydrographer in Geological Survey. May 11, 1903, appointed engineering aid in the reclamation service.

PRITCHETT, CHARLES MARCELLUS, assistant engineer. Address, Washington, D. C. Born in Georgia, August 10, 1869. Education, Georgia School of Technology (B. S., 1891); Agricultural and Mechanical College of Raleigh, N. C. (M. E., 1894, and C. E., 1895); also took special course in experimental engineering at Cornell University. Member of American Society of Civil Engineers. 1891–1900 assistant professor of engineering, Agricultural and Mechanical College. 1900–1902 with D. A. Tompkins

Company, Charlotte, N. C., in charge of surveys for development of water powers; preparing plans for cotton mills, cotton-seed oil mills, etc., and in charge of installation of machinery for cotton mill. April–August, 1902, with W. C. Whitner, Rock Hill, S. C. (chief engineer Catawba Power Company), in charge of survey for Catawba Power Company, and survey for railroad line from Rock Hill to Seaboard Railroad, also preparing plans for water-power development on Wateree River. August–October, 1902, with J. E. Sirrine, milling engineer, Greenville, S. C., in power department. October, 1902, to January, 1903, assistant engineer, United States Geological Survey, locating and determining available water powers in North Carolina and South Carolina. January 12, 1903, appointed assistant engineer in the reclamation service.

QUINTON, JOHN H., consulting engineer. Address, Los Angeles, Cal. Born in Ireland, October 19, 1850. Educated at Queens College, Belfast, and Queens College, Galway, Ireland. Degrees, B. A. and B. E., Queens University, Ireland. 1860–1866, at Portora Royal School, Ennis Killen, Ireland; 1866–1868, at Queens College, Belfast, Ireland; 1868–1871, at Queens College, Galway, Ireland. In 1873 spent six months as leveler on construction of a railway in Sligo, Ireland, and three months as leveler on Fresno River canal, California. In 1874 leveler on topographic survey of Burlingame tract near San Francisco, and leveler and transitman on Southern Pacific Railway, with which company he remained until June, 1876. In 1877 surveying and leveling in San Joaquin Valley, California. 1878–1880, assistant engineer in charge of location and construction on South Pacific Coast Railway, California. In 1880 engineer in charge of construction of 80 miles of Oregonian Railway. 1881–1884, assistant engineer, principal assistant engineer, and acting chief engineer Pacific Branch Mexican Central Railway. 1884–1888, in private practice as civil and hydraulic engineer in southern California. 1888–89, assistant engineer in War Department at Portland, Oreg. 1890–1892, field engineer for Hoffman & Bates, bridge builders, Portland, Oreg. 1892–93, principal assistant engineer in charge of location and construction of Santa Ana canal, California. 1894–1897, in private practice as consulting engineer in Los Angeles, Cal. 1898, engineer in charge of construction of San Gabriel power canal, including 36 tunnels and other works. 1899–1900, deputy city engineer in charge of construction of Third street and Broadway tunnels, Los Angeles. 1900–1902, in private practice in Los Angeles, Cal. 1902–3, consulting engineer with United States Geological Survey. May 5, 1903, appointed consulting engineer in the reclamation service.

REDMAN, ARVILLE C., assistant engineer. Address, Lawton, Okla. Born in Belfast, Me., January 16, 1877. Graduate of Massachusetts Institute of Technology; B. S., 1900. June, 1900, to July, 1900, draftsman for Edward L. Raymond, landscape architect, Boston, Mass. July, 1900, to June, 1903, employed on sewage-disposal system, Washington, D. C., designing, estimating, and construction work. June 1, 1903, appointed assistant engineer reclamation service.

REDMOND, HUGH, engineering aid. Address, Livingstone, Ariz. Born in New York State, October 18, 1880. Education, common school, high school, Cornell University; degree C. E. Specialized in hydraulic engineering at Cornell. During vacations from college did civil engineering work in farm surveys, relocating lines and laying out roads and building sites. During summer of 1902 assisted in surveys for dam on Sixmile Creek, for a pipe line from dam to pumping station, survey of site for pumping station, and pipe line connecting pumping station with the city mains. Also assisted in survey of filter plant, including location; also survey of Tanghannock Creek for reservoir site, dam, and power station. May 23, 1903, appointed engineering aid in the reclamation service.

REED, HOWARD SAWYER, assistant engineer. Address, Fort Duchesne, Utah. Born in Maine, April 27, 1869. Education, common and high school in home town. 1885–1894, in mercantile business in Boston, Mass. 1894–1897, with Moore & Co.,

Boston, engineers and constructors of waterworks on various projects throughout Maine and Massachusetts. 1897–1902, with the Nicaragua and Isthmian Canal Commission as assistant hydrographer, spending three years in hydrographic work on the Nicarargua route. 1902, assisting in the preparation of the report on the Salt River, Arizona, project, and the annual report of the hydrographic division of the United States Geological Survey. June 28, 1902, appointed resident hydrographer for work on the Uinta Indian Reservation, and May 28, 1903, assistant engineer in the reclamation service.

REED, WENDELL M., engineer. Address, Roswell, N. Mex. Born in Moriah, N. Y., December 28, 1864. Graduate of Sherman Collegiate Institute, Moriah, N. Y., and of University of Vermont, receiving degree of C. E. at latter in 1886. Has practiced irrigation engineering since 1889. 1889–1893, assistant engineer Pecos Irrigation and Improvement Company, Carlsbad, N. Mex. In 1893 was resident engineer on construction of distributing reservoir and made surveys and plans for Hondo reservoir dam, headworks, canals, etc., for the same company. In 1894 became engineer and water superintendent of Roswell Land and Water Company, Roswell, N. Mex., and retained this position until 1898, when he became chief engineer for Pecos Irrigation and Improvement Company, which position he held until 1900. During this time acted as expert for the Rio Grande Dam and Irrigation Company in their suit against the Department of Justice concerning the navigability of Rio Grande and the impounding of waters of Rio Grande in New Mexico. In 1900 established office in Roswell, N. Mex., for general irrigation engineering practice.

REEDY, OLIVER THOMAS, assistant engineer. Address, Livingstone, Ariz. Born March 10, 1873, at Nebraska City, Nebr. Graduate of University of Nebraska in C. E. 1898–1900, with Union Pacific Railroad as chainman, rodman, assistant draftsman, and draftsman. 1900–1902, in engineering department of United States military government of Cuba as topographical draftsman in charge of office work by the subdepartment survey of fortifications. July–September, 1902, with Mexican Central Railway as topographer and transitman. October 17, 1902, appointed assistant engineer in reclamation service and assigned to topographic work in Salt River Valley, Arizona. In October, 1903, assigned to duty as locating engineer on power canal.

ROBBINS, SAMUEL B., engineer. Address, Great Falls, Mont. Born in Connecticut, October 15, 1867. Private school, high school, and college education; graduate of Sheffield Scientic School of Yale. In 1886 was chainman with Union Pacific Railway. In 1887 was with Burlington and Missouri River Railroad as rodman, leveler, and assistant engineer. 1887–88, assistant engineer on Poughkeepsie Bridge system. August–November, 1889, masonry engineer on Lehigh Valley Railroad; then with Union Pacific Railway as assistant engineer on surveys till April, 1890; then in charge of construction on Ogden-Butte line, Bear River Canyon to Deweyville; afterwards in charge of construction between Portland and Seattle (work uncompleted). In June, 1891, opened civil-engineering office in Great Falls, Mont., making a specialty of irrigation work. Continued in general engineering practice until June, 1897, when he was employed by Boston and Montana Mining Company, at Butte, in charge of water supply and construction work. Resigned August 1, 1898, to take position as mine superintendent. Resigned October 1, 1899, and returned to Great Falls and resumed private practice, having charge of construction and making surveys on several large reservoirs and irrigation canals. July–October, 1902, made reconnaissance for reclamation service in central Montana. June 8, 1903, appointed engineer in the reclamation service.

ROSS, DOUGLAS WILLIAM, engineer. Address, Boise, Idaho. Born in Ontario, Canada, December 7, 1865. Education, Collegiate Institute, St. Thomas, Ontario, 1881–83. First engineering work, 1883–84 on public surveys of Canada on North and South Forks of Saskatchewan River, Territory of Alberta. 1886–1889, instrument man and topographer on railroad construction and location. 1900, topographer

United States Geological Survey and in charge of location of New York canal in Boise Valley, Idaho. 1891, in charge of maintenance of Phyllis canal in Boise Valley. 1892, resident engineer in charge of construction of Sunnyside canal in Yakima Valley, Washington. 1893 in charge of construction of Riverside canal in Boise Valley. 1893–1896, in general charge of operation of canal system of Payette Valley Irrigation and Water Power Company, Idaho. 1896–1898, manager hydraulic mining operations of Pioneerville Gravel Gold Company, in Boise Basin, Idaho; also engaged during same time in investigation of irrigation projects in southern Idaho. March, 1899, to March, 1903, State engineer of Idaho, and besides discharging duties of that office had charge during 1900, 1901, and 1902 of investigations determining the duty of water in irrigation in Idaho, under United States Department of Agriculture. July, 1901, to July, 1902, conducted reservoir and canal investigations in cooperation with United States Geological Survey, and since carrying on these investigations under reclamation service.

RUSSELL, WILLIAM GARDNER, hydrographer. Address, Russell, Kans. Born in Ohio May 17, 1850. Education, common schools, Plains Seminary, commercial college, student of civil engineering at National Normal School, Lebanon, Ohio; mining, electrical, and stationary engineer courses in correspondence school. 1868–1871, 1875–1878, was teacher, principal, and superintendent of schools. 1878 admitted to bar of Ohio and practiced law until 1882. Admitted to bar of Kansas in 1883. 1885–1903, surveyor of Russell County, Kans. 1892–1902, bridge engineer of Russell County, Kans. 1895–96, engineer of Kansas State board of irrigation. 1895–1902, hydrographer, United States Geological Survey. September 15, 1902, appointed hydrographer in the reclamation service.

SANDERS, WILLIAM HORATIO, consulting engineer. Address, Los Angeles, Cal. Born in Scotland July 12, 1842. Educated in Edinburgh, Scotland. 1886–1887, general engineering practice, chiefly in northern Minnesota. 1887–1903, hydraulic and sanitary engineering in southern California as follows: Ten years engineer Pomona Land and Water Company, water development and irrigation; chief engineer San Antonio Light and Power Company, designing and constructing the historical San Antonio power plant; consulting engineer for South Riverside Land and Water Company, also Santa Ana and Anaheim Water Company; designed and constructed system of sewer and septic tanks for city of Pomona, Cal. 1899–1903, chief engineer for San Antonio Water Company, irrigation works; designed and constructed Ontario power plant. May 15, 1903, appointed consulting engineer in reclamation service.

SANDS, EDWARD EMMET, assistant engineer. Address, Sparta, Wis. Born in Ohio, January 5, 1877. Graduate of University of Wisconsin; B. S. in C. E. July–November, 1898, rodman with United States Geological Survey. June–November, 1899, levelman, United States Geological Survey. 1900–1902, instructor in civil engineering at University of Wisconsin; also in charge of gaging of Wisconsin River. During the same period was also city engineer of Sparta, Wis., and designed sewer system and supervised construction of sewers and pavements. July–September, 1902, in charge of survey of an electric road from Sparta to Melrose, Wis. August, 1902, had charge of survey of Tomah, Wis., and designed sewer system and established grades. September 8, 1902, appointed assistant engineer in the reclamation service.

SAPH, AUGUSTUS VALENTINE, assistant engineer. Address, Reno, Nev. Education, University of California, B. S., 1894, and M. S. 1896; Cornell University, M. C. E., 1901; Ph. D., 1902. 1896–1900, instructor in drawing, University of California. 1900–1902, engaged in experimental work at Cornell University on resistance to flow of water in pipes, details of which were published in Transactions of the American Society of Civil Engineers, vol. 47, April, 1902, p. 295, and Proceedings American Society of Civil Engineers, May, 1903, p. 419, both of these articles having

been written in collaboration with Mr. E. W. Schoder. In 1902 assisted in making calculations for a proposed dam at Ithaca, N. Y. 1902–3, instructor in civil engineering at University of California. May 25, 1903, appointed assistant engineer in the reclamation service.

SARGEANT, WILLIAM, engineer. Address, Reno, Nev. Born in England, December 31, 1861. Education, English grammar school, finished in Neuwied, Germany. 1886–1888, draftsman on city, topographic, and general surveying in southern California. 1888–89, assistant engineer under War Department, at Portland, Oreg. 1889–90, on preliminary railroad surveys in Wasco County, Oreg. 1890–1892, topographer on railroad lines in Washington and Oregon. 1892, on canal location and construction in southern California. 1893–94, engineer in charge of relocation and construction of canal, Crafton Water Company. 1894–95, draftsman. 1895–96, superintendent Palm Valley Water Company. 1896–1902, water distributor and engineer under receiver for B. V. I. Co. 1902, field assistant, United States Geological Survey. May 25, 1903, appointed assistant engineer reclamation service, and on November 25, 1903, promoted to engineer.

SARGENT, JOSEPH ANDREWS, assistant engineer. Address, office of district engineer, Chamber of Commerce Building, Denver, Colo. Born at Escanaba, Mich., September 29, 1872. Graduate B. S. in C. E., University of Nebraska. Associate member American Society of Civil Engineers. From 1888 to 1898, when not in college, served variously as axman, rodman, chain man, leveler, topographer, and transit man on railroad location, construction, maintenance of way, freight dock construction, and Mississippi River incline building. Served during Spanish-American war as sergeant, Second United States Volunteer Engineers, on camp sanitation and water supply. Had charge of party of engineer soldiers on preliminary survey of Havana and vicinity. 1899–1902, as assistant engineer to the chief engineers of United States military government of Cuba, organized and directed fortification and topographic surveys of Havana. October, 1901, to April, 1902, also organized and completed a reconnaissance survey of the island of Cuba. July–October, 1902, division engineer on construction, Mexican Central Railroad. December 3, 1902, appointed assistant engineer reclamation service.

SAUNDERS, HENRY J., engineering aid. Address, Washington, D. C. Born in Iowa, January 14, 1880. Graduate of University of Wisconsin, C. E. In 1899, with Union Pacific Railroad. 1899–1900, with city engineer, Madison, Wis. 1900, with Union Pacific Railroad in Utah and Wyoming. 1900–1901, on surveys of Lakes Mendota and Kegonsa, Wisconsin. 1901, on survey for water-supply system for Cumberland, Wyo. 1901–2, with Union Pacific Railroad in Kansas. 1903, with Union Pacific Railroad as assistant engineer on double track construction. January 21, 1904, appointed engineering aid in reclamation service.

SAVAGE, HIRAM NEWTON, consulting engineer. Address, National City, Cal. Education, New Hampshire College of Agriculture and Mechanical Arts, B. S.; Thayer School of Civil Engineering, C. E. In 1888 with Nashville and Tellico Railway; also same year engineer to Athens (Tennessee) Improvement Company and engaged in field work connected with location and construction of streets, street railways, and municipal water system for Athens. 1889–90, assistant engineer and chief engineer for Hydraulic Mining and Irrigation Company, operating in New Mexico. In 1890 engineer and superintendent of construction, Billings Park and race track, White River Junction, Vt.; also designed and located sewerage system for West Randolph, Vt. 1891 to 1903, chief engineer San Diego Land and Town Company. 1891, engineer in charge of maintenance and operation of Sweetwater dam, San Diego, Cal.; chief engineer National City and Otay Railroad Company, California. 1893, designed and superintended construction of Sweetwater Park and race track, National City, Cal. 1895 designed and superintended construction, repairs, and additions increasing capacity of Sweetwater dam. 1898, designed, installed pumps for, and has

since supervised distribution of an emergency supply of water under Sweetwater system. 1893–1898, consulted frequently by Southern California Mountain Water Company; then appointed permanent consulting engineer for that company. 1898 to date, consulting engineer, general practice; chief engineer San Diego, Pacific Beach and La Jolla Railroad Company, and chief engineer San Diego, Cuyamaca and Eastern Railroad Company, California. 1901–2 maintenance, etc., San Diego Flume Company; on semitranscontinental railway from El Paso, Tex., to Pacific coast; associate engineer San Diego Eastern Railway project. 1902 investigations, etc., Otero irrigation district, Colorado. 1903 material contractor's engineer for United States Government jetty under construction at entrance to San Diego Bay. August 1, 1903, appointed consulting engineer in the reclamation service.

SAVAGE, JOHN LUCIAN, assistant engineer. Address, Madison, Wis. Born in Wisconsin December 25, 1879. Graduate of University of Wisconsin, B. S. in C. E. May 11, 1903, appointed engineering aid in reclamation service, and assigned to duty in the State of Idaho; April 1, 1904, promoted to assistant engineer.

SAWYER, WILBUR CYRUS, engineering aid. Address, Washington, D. C. Born in Vermont May 8, 1877. Graduate of University of Vermont, B. S. in C. E. From 1900 to 1903 was instructor in surveying and drawing at University of Vermont. Summers of 1900 and 1901, draftsman on steel construction, New York City. July 10, 1903, appointed engineering aid in the reclamation service.

SAXTON, WILLARD ROY, engineering aid. Address, Berlin, Wis. Born in Wisconsin November 6, 1881. Graduate of University of Wisconsin, B. S. in C. E. In 1902, level man on topographic work in North Dakota, under United States Geological Survey. June 11, 1903, appointed engineering aid in the reclamation service.

SCHADLER, AUGUST H., engineering aid. Address, Reno, Nev. Born in Germany March 8, 1879. Graduate of Nevada State University, B. S. In 1902 made stream measurements and ran levels in Nevada for United States Geological Survey. April 20, 1903, appointed engineering aid in the reclamation service.

SCHLECHT, WALTER WILLIAM, assistant engineer. Address, 10 River View Terrace, New York, N. Y. Born in New York City, October 27, 1874. 1891–1897, at New York City College and Columbia University. Graduated in 1897 as civil engineer. 1897–1901 with Nicaragua and Isthmian Canal commissions in Nicaragua, Panama, and Washington, D. C., in charge of hydrographic investigations under A. P. Davis, chief hydrographer. 1901–2 resident hydrographer, United States Geological Survey, at Kingston, N. Y. July 3, 1902, appointed assistant topographer in the reclamation service; January 1, 1904, designation changed to assistant engineer.

SLICHTER, CHARLES SUMNER, consulting engineer. Address, Madison, Wis. Born in Minnesota, April 16, 1864. Education: Four years' science course and one year graduate study at Northwestern University; B. S. and M. S. Northwestern University; research work while professor of applied mathematics in University of Wisconsin since 1886. Since 1894 has done special work for United States Geological Survey on motions of underground waters. Has published many books and papers on scientific and technical subjects which have been printed in American scientific and engineering periodicals. Devised and put in use in the field in 1901 a new method of measuring the velocities of ground waters. Has been called in consultation on ground waters by United States Department of Agriculture, Santa Fe Railroad Company, and others. Has undertaken the study of the theory of the plastic solid, which has for one of its objects the determination of the distribution of stress in earthen dams, and for this purpose has built apparatus for magnifying the rate of motion of the particles of plastic solids by the factor 100,000, which apparatus is the basis of the experimental study of the distribution of stress in such bodies. January 1, 1903, appointed consulting engineer in the reclamation service.

SLONAKER, CHARLES EDWARD, engineering aid. Address, Los Angeles, Cal. Born in Pennsylvania, October 27, 1877. Graduated from public schools of New Kingston

and from the preparatory school of Dickinson College. In 1896–97 taught a public school. In 1901 was appointed observer in the United States Weather Bureau, and on April 20, 1903, was transferred to the reclamation service.

SMITH, CHARLES BAILEY, assistant engineer. Address, Boise, Idaho. Born in Michigan, May 11, 1879. Education: Agricultural College of Utah, B. S. in C. E. From May, 1901, to April, 1902, in charge of field party for the Telluride Power Company, Provo, Utah. May, 1902, to September, 1902, inspector and division engineer with Southern Pacific Railroad at Fenelon, Nev. September, 1903, to October, 1903, engineer in charge of field work on Glenns Ferry Canal, Bliss, Idaho. December, 1902, to February, 1903, draftsman in office of United States surveyor-general, Boise, Idaho. March 1, 1903, to May 26, 1903, draftsman with reclamation service. Appointed assistant engineer May 27, 1903.

SMITH, WALTER D., engineering aid. Address, Redlands, Cal. Born in California, 1878. High school education. 1889–1902, assistant city engineer, Redlands, Cal. Appointed engineering aid in reclamation service February 1, 1903.

SOUTHWORTH, CLAY HARFORD, engineering aid. Address, Reno, Nev. Born in Canada, September 4, 1881. Education: High school; University of Nevada, degree B. S. From July to September, 1900, worked in cyanide plant near Bodie, Cal. July–September, 1901, draftsman for Sweetwater Consolidated Mining and Milling Company, making plans for cyanide plant; during part of this time worked on surveys. Has been employed with United States Geological Survey as rodman, levelman, and transitman. May 11, 1903, appointed engineering aid in the reclamation service, since which date has acted as plane-table man and chief of party.

SPRIGG, ALDEN M., assistant engineer. Address, Phoenix, Ariz. Born in Maryland, November 16, 1857. Attended private schools in Cumberland; St. James College, Hagerstown; St. Johns College, Annapolis, and Johns Hopkins University, Baltimore. 1877–78, land surveying as chainman in Maryland. 1878–79, manager of plantation in Mississippi. 1879–80, on construction of railroad in western Maryland as rodman and chainman. 1880–81, with Baltimore and Ohio Express Company, Baltimore, and Fidelity Title and Trust Company, same place. 1881, rodman and chainman in engineering department, Baltimore and Ohio Railroad. Also, in same year, on various railroad surveys in West Virginia and Pennsylvania as rodman, instrument man, and in charge of party. 1881–1885, mine surveys for Consolidated Coal Company in Maryland as transitman and draftsman. 1883–1902, with Baltimore and Ohio engineering department as transitman, assistant engineer, and division engineer. 1902–3, on street-railway construction and laying out lots in Pittsburg, Pa. May 28, 1903, appointed assistant engineer in the reclamation service.

STABLER, HERMAN, hydrographic aid. Address, Washington, D. C. Born in Maryland, 1880. Graduate Earlham College, Richmond, Ind., B. S. Course in irrigation engineering at Columbian University, Washington, D. C., leading to degree C. E. 1899–1903, instructor in mathematics, Washington, D. C. December 30, 1903, appointed hydrographic aid in reclamation service.

STANNARD, JAY D., assistant engineer. Address, Denver, Colo. Born in Michigan, July 8, 1855. Graduate of Michigan Agricultural College, 1876. 1876–1888, farming and surveying in Michigan and Wisconsin. 1888–1891, county surveyor and city engineer, Greeley, Colo. 1891–1900, assistant in department of civil and irrigation engineering, Colorado Agricultural College, in charge of classes in physics, surveying, irrigating engineering, strength of materials, roofs and bridges, and drawing. 1900–1903, assistant in irrigation investigations, United States Department of Agriculture; was assigned to studies on Weber River, Utah; Humboldt River, Nevada; and Snake, Big Lost, Big Wood, and Little Wood rivers, Idaho. April 16, 1903, appointed assistant engineer in the reclamation service, and assigned to work on the Sweetwater project, Wyoming.

STEINER, CHARLES RODOLPH, assistant engineer. Address, Phoenix, Ariz. Born in

Switzerland, October 2, 1857. Education: Three and one-half years at Zurich Polytechnic School, Switzerland. 1879–1882, city engineer's office, Zurich, Switzerland. 1882–1884, bridge, structural, and hydraulic engineering with two concerns in France. 1885–86, two years of travels to India and various countries of Europe and South America for power plants, harbor construction, and railway projects. 1887–88, in Germany and Switzerland, eighteen months on factory designs, and remainder of time with railroad company and on military work. 1889–1902, with interruptions, draftsman, designer, translator, computer, checker on structural and hydraulic work with consulting engineers, bridge and structural concerns, railroad and power companies in the United States. June 5, 1903, with reclamation service as expert designer and assistant engineer.

STEVENS, JOHN CYPRIAN, assistant engineer. Address, Station A., Lincoln, Nebr. Born in Kansas, January 9, 1876. June, 1902, to November, 1903, was assistant hydrographer for United States Geological Survey, on stream measurements, etc., in Nebraska. August, 1902, to February, 1903, agent and expert in irrigation investigations for Department of Agriculture. November, 1902, to March, 1903, assistant State engineer of Nebraska. March 1, 1903, appointed assistant engineer in the reclamation service.

STEWARD, WILLARD G., assistant engineer. Address, Livingstone, Ariz. Born in California, March 3, 1861. Graduate of Michigan State Normal School; studied mechanical drawing at Toulane University, New Orleans; mechanical course at Michigan Agricultural College; C. E., 1891, at Columbian University, Washington, D. C. From 1885 to 1887 was teacher, superintendent, and principal of schools. 1889–1894, in Record and Pension Office, War Department. 1894–1897, Pension Office, Interior Department. 1897–98, photographic and mechanical expert with American Parlor Kinetoscope Company. 1898–1902, Pension Office, Interior Department. 1892–1897 was with engineer corps, District of Columbia National Guard. October 10, 1902, appointed engineering aid in the reclamation service. July 1, 1903, promoted to assistant engineer.

STOCKMAN, LOUIS RICHARD, assistant engineer. Address, Milwaukee, Wis. Born in Wisconsin, November 7, 1876. Graduate of University of Wisconsin, degree B. S. In 1897 and 1898 taught school. During summers, while attending college, worked on farm. In 1901, was levelman for United States Steel Company. In 1902, with K. LaF. & J. R. R., in Tennessee; also, August–November, with Louisville Property Company. 1902–3, with United States Geological Survey. May 5, 1903, appointed assistant engineer in the reclamation service.

STOCKTON, FRED, engineering aid, Boise, Idaho. Born in Missouri, December 21, 1880. Graduated, University of Oregon, 1903, B. S. May–September, 1899, on contract surveying. June–November, 1902, with C. M. Collier, engineer, Eugene, Oreg., doing general surveying and engineering. November, 1902, to June, 1903, devoted half time to work with Mr. C. M. Collier. Appointed engineering aid, United States Geological Survey, June 6, 1903.

STOCKTON, ROBERT SUMMERS, engineer. Address, Chamber of Commerce Building, Denver, Colo. Born in Illinois June 5, 1872. Graduate of Colorado School of Mines, degree E. M. In 1892 worked on Government subdivision survey near Pagosa Springs, Colo. In 1894 engaged on Government subdivision survey near Delta, Colo. In summer of 1896 worked on Madison River, in Montana, for the Power Company. During the summers of 1895 and 1897, in the Cripple Creek district, prospecting and studying. Summer of 1899, employed with the engineering force engaged in enlarging the power plant on the Provo River, Utah, for the Telluride Power Company. Summer of 1902, superintendent of power transmission plant of the Telluride Power Company, on the Provo River, Utah. 1895–1903, professor of mathematics and surveying at Colorado School of Mines. June 5, 1903, appointed engineer in the reclamation service.

STOKES, HUGH GREGORIE, engineering aid. Address, Denver, Colo. Born in South Carolina July 25, 1879. Graduate of Clemson College, degree B. S.; course in mechanical and electrical engineering. From August 1 to November 1, 1902, employed by the United States Geological Survey as traverseman and topographer; from January 10 to July 10, 1903, as field assistant, making stream measurements and recording observations or gage readings, and on July 10, 1903, appointed engineering aid in the reclamation service.

STORRS, HARRY ASAHEL, electrical engineer. Address, Chamber of Commerce Building, Denver, Colo. Born in Boston, Mass., January 13, 1861. Graduate of University of Vermont, degree C. E., in 1882. In 1882–83 was on railway work in Ohio and West Virginia. 1883–84, superintendent of waterworks at Burlington, Vt. 1884–85, on topographic work in Virginia. 1885–1887, with Western Electric Company, New York and Chicago; became electrician at New York and had charge, among other projects, of laying cables from Nantucket to mainland and from Cape Charles to Cape Henry for the Signal Corps. In 1887 was chief engineer for Pomona Land and Water Company, California; had charge of making and laying cement pipe, of building dams, flumes, reservoirs, etc., and of surveys for water-power development, etc., in the Sierra Madre Mountains. In the summer of 1888 laid the Government cable across mouth of Columbia River for the Western Electric Company. Left California in 1890, and after a few months in New York in electrical work, accepted position of assistant professor of civil engineering in University of Vermont. Pursued post-graduate studies at Columbia University in 1893–94. In 1892–1899 was professor of electrical engineering in University of Vermont, during which time acted as consulting electrical and hydraulic engineer on electric plants in Vermont. In the summer of 1893 did the instrumental work on a survey of St. Lawrence River under United States Engineer Corps. In 1897 was on construction work on seacoast fortifications, including electrical installations, and during summer of 1898 on the same, and was also chief electrician of harbor mines during Spanish war. 1899–1903, electrical engineer under the War Department, designing and installing electric plants; also on river and harbor improvements. Member of American Institute of Electrical Engineers, and Connecticut and Boston societies of civil engineers. April 18, 1903, appointed electrical engineer in the reclamation service.

STRATTON, GEORGE E., assistant engineer. Born in Massachusetts June 2, 1873. Education, Massachusetts Institute of Technology, C. E. course. During summers 1891–1893 was rodman, transitman, and inspector at Brookline, Mass. Summer of 1894, rodman in Boston sewer department. Summer of 1895, rodman in Boston sewer department, in charge of field party on surveys and construction. 1896–1898, principal transitman with Metropolitan Sewer Commission, Boston, Mass. 1898–1903, junior engineer, War Department, Boston district. Member Boston Society of Civil Engineers; associate member American Society of Civil Engineers. June 13, 1903, appointed assistant engineer in the reclamation service.

STRICKLER, THOMAS JOHNSON, engineering aid. Address, 3428 Campbell Street, Kansas City, Mo. Born in Kansas, May 21, 1883. Education, civil engineering work at Wentworth Military Academy and Kansas State University. During summers 1899–1901, assistant manager lead and zinc ore producing plants at Galena, Kans., and Joplin, Mo. Summer and fall 1902, assistant division engineer on railroad construction, Mexican Central Railroad. Spring, 1903, recorder in reclamation service, Colorado River project. June 10, 1903, appointed engineering aid in reclamation service.

SWENDSEN, GEORGE LEWIS, engineer. Address, P. O. Box S., Salt Lake City, Utah. Born in Utah December 21, 1869. B. S. in C. E., Harvard University, 1894. During the summer of 1893 was in charge of a party in Boston City Board of Surveys. 1900–1901, engineer of construction for aqueduct and pipe lines for Hercules Power Company, Logan, Utah. 1901–2, employed by the Last Chance Canal Company,

Cove, Idaho, as engineer of dam in Bear River and diversion canal. April–July, 1902, employed by North Extension Canal Company, Turner, Idaho, as engineer. June–August, 1902, engineer of projected reservoir at the head of Cub River, for Cub River Reservoir Company, Lewiston, Utah. August–September, 1902, in charge of party investigating Bear Lake as a storage reservoir for United States Geological Survey. Fall of 1902, consulting engineer for aqueduct pipe line and canal of Glenns Ferry Land and Irrigation Company, Glenns Ferry, Idaho. 1902–3, consulting engineer in subdivision of water of Logan River, Logan, Utah. May 1, 1899, to February 7, 1903, professor of civil engineering in Utah Agricultural College. 1899–1903, resident hydrographer, United States Geological Survey. February 7, 1903, appointed engineer in the reclamation service.

SWIFT, WILLIAM EVERETT, engineer. Address, Wadsworth, Nev. Born in Connecticut January 1, 1871. B. S., Massachusetts Institute of Technology. June–August, 1895, rodman in engineering department, Boston waterworks. 1895–1899, with Metropolitan Waterworks, Boston, as rodman, instrumentman, and head of party. 1899–1900, with Munson Steamship Line, New York City. 1900–1903, assistant engineer, Rapid Transit Railroad, New York City, in charge of construction of section of subway under Forty-second street and Broadway. February 24, 1903, appointed engineer in the reclamation service.

TANNER, CALEB, assistant engineer. Address, P. O. Box S, Salt Lake City, Utah. Born in Utah September 3, 1869. Graduate of Lawrence Scientific School, degree, S. B. University education at Harvard. 1896–1903, instructor in mathematics and surveying in the Brigham Young Academy, Provo, Utah. From 1897 to 1903 was city engineer of Provo, Utah. At intervals during summers of 1901–1903, hydrographer for United States Geological Survey. In 1902, measured the flow and investigated the seepage in the Sevier Valley, Utah. February 18, 1903, appointed assistant engineer in the reclamation service.

TAYLOR, CLYDE VICTOR, assistant engineer. Address, Reno, Nev. Born in California January 26, 1878. Education, University of California. 1900–1902 with hydrographic branch of United States Geological Survey, making stream measurements on Truckee, Carson, and Humboldt rivers and their tributaries; in charge of parties on surveys of reservoir sites and canal lines on those rivers in 1901 and 1902. Has also surveyed ditch lines and mining claims, and retraced Government subdivision of land lines and surveyed two townships in Elko County, Nev. January 1, 1903, appointed assistant engineer in the reclamation service.

TAYLOR, LEON HENRY, engineer. Address, Reno, Nev. Born in Comanche, Tex., October 2, 1863. Common and normal school education. For eight years resident hydrographer for United States Geological Survey, engaged in measuring discharge of streams, surveying and computing area and capacity of reservoir sites, planning and estimating cost of reservoir, and diverting dams and weirs. During this time was also employed on survey and plans for reservoir on Rock Creek, and canal system for irrigation of 100,000 acres of land in Humboldt River Valley; also on surveys and plans for two reservoirs in lower Humboldt River Valley, Nevada. Employed at different times from 1886 to April, 1892, inclusive, by various land and water companies in California as irrigation engineer, and by Francis G. Newlands for Sharon estate as engineer on location and designs of canal from Truckee River, California and Nevada, to irrigate 75,000 acres of land, May to August, 1892. Appointed engineer in reclamation service February 11, 1903.

TAYLOR, THOMAS U., resident hydrographer. Address, Austin, Tex. Born in Texas January 2, 1858. Graduate of University of Virginia, degree C. E., also M. C. E., Cornell University. Has been with United States Geological Survey since 1897; has had charge of all the streams in Texas, and has kept gaging stations on the Pecos, Brazos, Colorado, Trinity, and Guadalupe rivers; also made special measurements of nearly all the spring-formed rivers and creeks west of the Colorado,

and a number of those east of the river. Reports of these operations will be found in the Water-Supply Papers of the Geological Survey, special report on Austin dam water supply being published in Paper No. 40 of that series. Is author of a report on irrigation systems of Texas, published as Water-Supply Paper No. 71. January 1, 1903, appointed resident hydrographer in the reclamation service.

TEMPLE, FRED A., assistant engineer. Address, Reno, Nev. Born in Missouri August 10, 1871. Graduate of University of California, C. E., degree B. S. 1897–1899, was instrument man for the Los Angeles Railroad Company. 1899–1900, civil engineer for Independent Electric Light and Power Company, of San Francisco. 1900–1901, junior civil engineer at Mare Island Navy-Yard. 1901–1903, assistant engineer in city engineer's office of San Francisco, Cal. June 9, 1903, appointed assistant engineer in the reclamation service.

TILLINGHAST, FREDERICK HOWARD, assistant engineer. Address, Washington, D. C. Born in Providence, R. I., September 19, 1877. Graduate of Brown University, C. E. In 1899–1900 was special student in hydraulics and sanitary engineering at Massachusetts Institute of Technology. 1900–1901, instrument man for Chicago and Great Western Railway. May–November, 1901, assistant to J. F. Witmer, civil engineer, Buffalo, N. Y., making surveys for waterworks, sewers, etc.; also supervised construction of a waterworks plant, including power house and concrete reservoir. 1901–2, special agent for Equitable Fire and Marin Insurance Company, on map work. February–July, 1902, draftsman for Choctaw, Oklahoma and Gulf Railroad. July–October, 1902, assistant to superintendent on construction of masonry dam and power house, on Catawba River, for Catawba Power Company. October 9, 1902, appointed assistant engineer in the reclamation service.

TURNER, WILLIS T., topographer. Address, Phoenix, Ariz. Born in New Jersey, April 23, 1869. Three years at Vineland High School. In 1894 was station assistant, United States Geological Survey; 1895, traverseman. 1896–1898, United States surveyor, Indian Territory survey. 1898–99, field assistant, Pacific division, United States Geological Survey. 1899–1903, topographer, Pacific division, United States Geological Survey. June 15, 1903, transferred to reclamation service.

VEATCH, ARTHUR CLIFFORD, assistant geologist. Address, Washington, D. C. Born in Indiana, October 26, 1878. Graduate of Rockport, Ind., high school, 1895; student Indiana University, spring term, 1896. 1896–1898, teacher, Rockport High School. 1898, member Indiana University geological survey. January–April, 1898, student at Indiana University. April–June, 1898, student at Cornell University. June–September, 1898, member Cornell geological survey of the Tertiaries. 1898–1900, assistant geologist, Louisiana geological survey. 1900, assistant in charge of areal and stratigraphic geology, Cornell school of field geology. 1900–1901, student Cornell University, and assistant in stratigraphic geology. 1901–2 consulting geologist, Houston Oil Company. 1902, assistant in preparation of Louisiana geological report of 1902. 1902, professor of geology, Louisiana State University, and geologist, Louisiana geological survey. November, 1902–3, special assistant, hydrographic branch, in charge of investigation of underground waters in northern Louisiana and southern Arkansas; in charge of underground geology and water problems, Long Island investigation. September 20, 1903, appointed assistant geologist in the reclamation service.

VOORHEES, ISAAC SPURR, assistant engineer. Born in New York, September 18, 1872. High school and college education. Attended Polytechnic Institute of Brooklyn, B. S. in C. E. In 1900 did some surveying at Shelter Island, New York. 1900–1901 in employ of chief engineer of Central Railroad of New Jersey as draftsman and instrument man; also inspector in charge of hydraulic work. 1901–1903 in engineering department of Rapid Transit Railway Commission, New York City, as rodman and draftsman. January 7, 1903, appointed engineering aid in the reclamation service; January 1, 1904, promoted to assistant engineer.

WALTER, RAYMOND F., engineer. Address, Chamber of Commerce Building, Denver, Colo. Born in Illinois, October 31, 1873. Graduate of Colorado Agricultural College, degree B. S.; postgraduate course in same school, C. E. In 1894 went to Greeley, Colo., and since then has been actively engaged as irrigation engineer and surveyor. Was county surveyor of Weld County for four years and city engineer of Greeley, Colo., for three years; also assistant State engineer of Colorado for three years, having charge of canal ratings in irrigation division No. 1. From 1895 to 1903 was engineer for many of the large canal and reservoir companies in northern Colorado. May 20, 1903, appointed engineer in the reclamation service.

WARNER, GEORGE LITTLETON, engineering aid. Address, Washington, D. C. Born in Ohio, March 23, 1879. Completed half of a five-year course at Columbian University, Washington, D. C. 1896–1898, rodman, Suburban Electric Company, Akron Ohio. 1899–1900, transitman on city surveys, Washington, D. C. 1901, topographer on Washington and Annapolis Suburban Electric Railway, Washington, D. C. 1901–2, computer, hydrographic work, Maricopa County water storage commission; also, same year, computer, United States Geological Survey; also assistant in sewer division, engineer department of the District of Columbia. September 26, 1902, appointed engineering aid in the reclamation service.

WEISS, ANDREW, assistant engineer. Address, Golden, Colo. Born in Austria, July 18, 1867. Graduate of Colorado School of Mines, degree E. M. 1899–1903, teaching mathematics and surveying. During the summers of 1900 and 1901 was employed as mine surveyor by parties in Cripple Creek, Colo. During the summer of 1902 worked with party of Colorado and Southern Railway engineers on location of line of road; also on maintenance of way. June 4, 1903, appointed assistant engineer in the reclamation service.

WEYMOUTH, FRANK ELWIN, engineer. Address, Orono, Me. B. C. E., University of Maine in 1896; C. E., same university in 1899. 1896–97, sewer construction, Malden, Mass. 1897–1899, waterworks construction and surveys, Boston, Mass. January to August, 1899, assistant city engineer, Winnipeg, Manitoba, and resigned to accept position with Isthmian Canal Commission on surveys for a proposed oceanic canal through Nicaragua; was in Nicaragua August, 1899, to October, 1900. October, 1900, to May, 1901, in Washington on same work. May, 1901, to March, 1902, with the Guayaquil and Quito Railway Company, Ecuador, South America, as resident engineer on construction work. June, 1902, to January, 1903, with United States Geological Survey on surveys in Montana and in the Washington office.

WHISTLER, JOHN T., engineer. Address, Washington, D. C. Born at Newmarket, Ohio, in 1865. 1884–1891, student, teacher, and principal of schools. 1891–1893, with Arrowhead Reservoir Company, of California. 1894, prospecting in Cripple Creek, Colo. 1894–95, engineer for Cachavi Mining Company, in Ecuador, South America. 1895–1897, structural steel draftsman, designer, and engineer. 1897–98, in private practice in New York as structural engineer. 1898, draftsman and engineer for Hamilton Bridge Works Company, Hamilton, Canada. 1898–99, dry dock construction in Brooklyn, N. Y.; also with Nicaragua Canal Commission as draftsman and computing engineer. 1899, United States surveyor under direction of United States Engineer Corps, in hydrographic surveys about Staten Island and Sandy Hook waters and upper Hudson River. 1899–1902, with Isthmian Canal Commission. April to August, 1902, assistant engineer on preliminary surveys and location for Baltimore and Ohio Railroad in western Pennsylvania. August, 1902, to present time, with United States Geological Survey as engineer in reclamation service.

WHITE, THOMAS BROOK, assistant engineer. Address, Portland, Oreg. Born in England December 3, 1860. English public school and collegiate education, followed by a course of study under private tutor. Attended lectures at Heidelberg. Graduate of Union College of Law, Chicago, degree LL. B. 1880–1882, on Northern Pacific and Oregon Pacific railroads as draftsman, and later as assistant in charge of office.

1882–83, draftsman. 1888–1891, draftsman and clerk, engineer Portland waterworks. 1898–99, Oregon Railroad and Navigation Company as draftsman. 1899–1900, field work, topographic draftsman, Corvallis and Eastern Railroad, and on completion of survey took similar position with Goble, Nehalem and Pacific Railroad. In 1901, similar position with Oregon Short Line. May 1, 1903, appointed assistant engineer in the reclamation service.

WILEY, ANDREW J., engineer. Address, Boise, Idaho. Born July 15, 1862. Graduate of Newark College and of Delaware College, Ph. B. 1883–1889, on railroad and irrigation surveys. 1889–90, with United States Geological Survey on canal surveys in Snake River Valley. 1890–91, chief assistant engineer Idaho Mining and Irrigation Company. 1892–1899, chief and resident engineer Owyhee Land and Irrigation Company. 1899–1900, in private irrigation practice. 1900–1901, chief engineer, Trade Dollar Consolidated Mining Company. 1891–1903, in general hydraulic engineering practice. Designed and constructed plant of Boise-Payette Electric Power Company, also Crow Creek reservoir dam for the city of Cheyenne. Designed power plants for the McCloud River Electric Power Company, Boise, Idaho, and made preliminary plans for dam and headworks of Twin Falls Land and Water Company, on Snake River. April, 1904, appointed engineer in the reclamation service.

WILLIAMS, HAROLD S., engineering aid. Born in Wisconsin May 26, 1878. Graduate of Phillips (Wisconsin) High School; special course in engineering at Ripon College. In 1899 was rodman and chainman for various parties. In 1900, stakeman and levelman on railroad work. In 1901 was draftsman at Phillips, Wis.; in same year, assistant county surveyor, Phillips, Wis. In 1902 was rodman, chainman, and draftsman for various parties. September, 1902, to January, 1903, rodman in irrigation work, United States Geological Survey. January 8, 1903, appointed engineering aid in the reclamation service.

WILLIAMS, MELVIN D., assistant engineer. Born in Iowa August 5, 1876. Graduate of Colorado Agricultural College. In 1898 and 1899 was assistant engineer, West Gallatin Irrigation Company, Bozeman, Mont.; also, in 1899, on seepage measurements and stream gaging, Colorado Experiment Station, Fort Collins, and on same work under State engineer, Colorado, in 1899–1900. 1901, transitman, Woods Instrument Company; also, same year, gager, Colorado Experiment Station, and rating flumes for Holly Irrigation and Beet Sugar Company; also, same year, hydrographer under State engineer, Colorado. 1901–2, professor of irrigation engineering and drafting, Colorado Agricultural College. 1902, engineer, Homestead Irrigation Company, in Wyoming. September 17, 1902, appointed assistant engineer in the reclamation service.

WISNER, GEORGE Y., consulting engineer. Address, 34 West Congress street, Detroit, Mich. Born West Dresden, N. Y., July 11, 1841. Graduate of University of Michigan, C. E., 1865. 1865–1880, engaged in Government surveys on Great Lakes and Mississippi River. 1880–1884, on surveys of Mississippi, Des Plaines, and Illinois rivers improvements. 1884–1887, superintendent of construction, Tenth and Eleventh United States lighthouse districts. Since then has been engaged in private practice. 1888–1891, resident engineer South Pass Jetty works and chief engineer Brazos River Harbor improvement; 1895–1897, organized and directed sanitary department, Detroit Board of Health; 1897–1900, member United States Board of Engineers on Deep Waterways. Author of Geodetic Field Work, 1883; Brazos River Harbor Improvement, 1891; Worthless Government Engineering, 1892; Breakwaters, Sea Walls, and Jetties, 1893; Hydraulics of Rivers having Alluvial Beds, 1896; Sewage Disposal, 1896; Regulation of Lake Levels, 1895 and 1899; Report of United States Board of Engineers on Deep Waterways, 1900; Canals from the Great Lakes to the Atlantic, 1900. March 23, 1903, appointed consulting engineer in the reclamation service.

YOUNG, WILLIAM ERNEST, assistant engineer. Address, Salt Lake City, Utah. Born in Marion, Ill., December 3, 1869. Public school and collegiate education. Graduated A. B., Southwest Baptist College, Bolivar, Mo., 1889. 1889–90, rodman and transitman, location and construction G. & N. R. R. in Missouri. 1891–92, instrumentman and engineer Boise City and Nampa Irrigation Land and Lumber Company. 1892, instrumentman Boise, Nampa and Oyhee Railroad. 1893–94, engineer B. C. & N. I. L. & L. Co. 1894, instrumentman on survey for State of Idaho, Minidoka project; local engineer, irrigation ditches, Orchards' Farms Reservoir and Canal Company. 1895, business for self and B. C. & N. I. L. & L. Co. 1896, transitman Fort Hall Indian Reservation irrigation surveys; 1897, engineer; 1898, business for self. 1899, locating and track engineer Pacific and Idaho Northern Railroad. 1900, transitman Oregon Short Line Railroad in Idaho, Utah, Wyoming, and Montana location surveys. 1901, division engineer, construction Oregon Short Line Railroad. 1901–2, division engineer, construction St. Louis, Iron Mountain and Southern Railroad in Arkansas. 1902, division engineer, construction Oregon Short Line Railroad. 1902–3, reconnaissance surveys for United States on Flathead and Northern Cheyenne Indian reservations. May, 1903, appointed assistant engineer in reclamation service.

ZORN, GEORGE WASHINGTON, assistant engineer. Address, Hecla, Wyo. Education, high school, Iowa State College, degree B. C. E. In 1896 was rodman on United States Geological Survey. In 1897, levelman, irrigation canal in Wyoming; also same year rodman, United States Geological Survey, in Wyoming and Kansas. 1898, levelman on Iowa Central Railway; also same year levelman, United States Geological Survey. 1899, levelman, Iowa Central Railway. 1899–1901, chief engineer reservoir construction for Wyoming Development Company. 1901–2, general engineering work. 1902–3, resident engineer on construction of masonry dam for city of Cheyenne, Wyo. July 24, 1903, appointed assistant engineer in the reclamation service.

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UNITED STATES GEOLOGICAL SURVEY
CHARLES D. WALCOTT, DIRECTOR

HYDROGRAPHIC MANUAL

OF THE

UNITED STATES GEOLOGICAL SURVEY

PREPARED BY

EDWARD C. MURPHY, JOHN C. HOYT, AND
GEORGE B. HOLLISTER

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
HYDROGRAPHIC BRANCH,
Washington, D. C., February 1, 1904.

SIR: I have the honor to transmit herewith a manuscript entitled "Hydrographic Manual of the United States Geological Survey," and to ask that it be published as a water-supply and irrigation paper.

It gives instructions for field and office work relating to gaging of streams by the use of current meters. Instructions relative to gaging streams by the use of weirs and dams will be embodied in a future edition of this manual.

This manuscript has been prepared by a committee composed of Messrs. Edward C. Murphy, John C. Hoyt, and George B. Hollister. They have endeavored to bring together all available information in regard to the methods of gaging streams which have been developed by the engineers and hydrographers of the United States Geological Survey, and in so doing have, as far as possible, consulted these men.

The publication is intended mainly for those engaged in hydrographic investigations for the Geological Survey. It is believed, however, that engineers and others not connected with the Government service who are interested in hydraulic problems will find it of much assistance. It is hoped, also, that teachers of civil engineering will make use of it in their courses of instruction, so that young men who enter this branch of the Government service may be familiar with the stream-gaging methods herein set forth.

Very respectfully,

F. H. NEWELL,
Chief Engineer.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

HYDROGRAPHIC MANUAL OF THE UNITED STATES GEOLOGICAL SURVEY.

Prepared by E. C. MURPHY, J. C. HOYT, and G. B. HOLLISTER.

INTRODUCTION.

The problem presented to the United States Geological Survey when it started to make systematic stream measurements throughout the United States, was to obtain with a small sum of money a large amount of information concerning the principal rivers of the country. In order that the data obtained should be of most value it was decided that both the total yearly flow of the streams and the seasonal distribution should be ascertained. Methods of procuring data of this broad nature had not at that time been developed, and many engineers considered the task impossible. However, on careful analysis of the conditions, it was found that the data could be obtained, and that in order to procure them two factors should be determined—first, the stage of the stream from day to day; second, the discharge corresponding to the various stages. The hydrographers and engineers engaged in the work have spent much time in devising means for determining these factors, and as a result well-defined methods have now been developed.

As literature in regard to these methods is either widely scattered or entirely lacking, this manual has been prepared. Its object is two-fold—first, to act as a guide for the engineers and hydrographers employed by the Geological Survey; second, to give the engineering public the benefit of these studies, in order that the methods of determining the facts as well as the data obtained may be more widely and fully understood.

The work of gaging streams would be greatly simplified if a single rule or method could be given which, when carefully followed, would in all cases give the most satisfactory results, but owing to the great diversity of conditions in different sections of the country it has been found that no such rule can be given; therefore, an effort has been made to state simply the methods of performing the various operations with the conditions to which each is applicable.

During the last few years the practice of gaging streams by weirs and dams has come into use to a considerable extent in the northeastern part of the United States, but the treatment of this method has been left for a future edition of this manual.

It is requested that those using this manual will make any suggestions that they think will be of value, in order that such suggestions may be incorporated in future editions.

Acknowledgments.—Thanks are due to many engineers and hydrographers for aid in the preparation of this manual. In this connection special acknowledgments are made to Messrs. N. C. Grover, R. E. Horton, M. R. Hall, A. L. Fellows, M. C. Hinderlider, John E. Field, B. M. Hall, O. V. P. Stout, T. A. Noble, G. L. Swendsen, and F. W. Hanna.

FIELD OPERATIONS.

SELECTION OF GAGING STATIONS.

Classes and location of stations.—Gaging stations may be divided into two classes, temporary and permanent; the former are maintained for one season, the latter for a series of years. Permanent stations should be selected only after a very thorough reconnaissance, so that the best results for the river and section investigated may be obtained, without breaks in the record. In the eastern part of the United States data are wanted mainly for water-supply and power purposes; in the central part, for water-supply and sanitary purposes; and in the West for irrigation and domestic purposes. At some stations in each section of the country information is desired for general statistical uses. The sanitary work carried on in connection with stream-gaging work consists mainly in determining the degree of dilution of sewage in the streams, and this work is done to some extent in the eastern as well as in the central part. Each station should be located so as to secure the requisite data with the proper degree of accuracy and at reasonable cost. For power purposes data concerning low and ordinary stages are more valuable than data for higher stages; hence low-water conditions should govern the selection of stations. Where the information is to be used mainly to determine the feasibility of storage projects, a station should be so located that high as well as low water can be measured with accuracy. Where the information sought is for irrigation purposes stations should preferably be established above all diversions, at points reached by telephone, or at such points as will aid in the distribution of the water.

Favorable conditions for current-meter gaging stations.—The channel at a gaging station should be as nearly straight as possible for from 200 to 500 feet above and below the station, the distance depending on the size of the stream, and there should be few if any obstructions.

The bed should be fairly permanent, regular in shape, and have few projections of more than 4 inches above its general coutour. There should be no sudden change in velocity, and the velocity should not be less than one-half foot per second in more than 15 per cent of the cross section. The station should be far enough above the junction with other streams to be free from the influence of backwater and should be beyond the influence of dams. The banks should be fairly high and not liable to overflow, except during high floods. It should be easily accessible and there should be a reliable gage reader located within a quarter of a mile of the gage.

Unfavorable conditions for current-meter gaging stations.—A gaging station should not be established where a reliable gage reader can not be secured; at a bend in a stream; at a bridge of short spans where drift collects or where the sides of the piers are not approximately parallel to the stream; at high trestle and railway deck bridges, on account of danger; at covered bridges, unless provided with numerous windows; near the mouth of a river having little fall; within the backwater above a dam; nor within such a distance below a dam that the shifting of currents in the stream channel, caused by the flow or cessation of flow over the spillway or through the turbines, has not disappeared. A sandy, shifting section is to be avoided if possible, as a rating curve for such a station is applicable for only a limited time. Sand beds or bars adjacent to the gaging section, which by shifting or scouring might change the velocity in the section, should be avoided.

It frequently happens that during the higher stages good results can be obtained from a bridge, but during the lower stages the flow becomes too sluggish or the depth too shallow to permit accurate measurement. Very often the best results can be obtained by wading, or by the use of a boat, at places not far from the station.

Whenever possible a station should be situated a short distance above rapids or a place of permanent bed. The rapids themselves seldom offer a good location, for the stream there is likely to be very rough and shallow and the velocity high. If the station is located too far above the rapids the stream is likely to be sluggish at low stages. The scouring or filling above the rapids has little effect on the station rating curve.

CLASSIFICATION AND EQUIPMENT OF GAGING STATIONS.

Kinds of stations and items of equipment.—Current-meter gaging stations may be divided, according to equipment, into bridge, cable, and boat stations. The equipment of a station consists of a bridge, or a cable and car, or a boat, as the case may be, from which measurements are made; a tag line and tags or marks on the bridge indicating the points at which the meter is lowered and soundings taken; a gage

for reading the surface fluctuations; bench marks for fixing the elevation of the zero of the gage; and when high velocities are to be measured a stay line for keeping the meter in place (see fig. 2).

Bridge stations.—A bridge station is preferable to either a cable or a boat station, if the conditions are good, on account of greater accessibility, lower cost of maintenance, and ease and rapidity with which the measurements may be made. The measurements made at a bridge are not, as a rule, as accurate as those made from a cable or boat, for conditions are not likely to be so favorable.

Cable stations—(*See Pl. I, A*).—In case a bridge station is not available and the span is less than 500 feet, a cable can be stretched across the stream at right angles to the current at a point where conditions are satisfactory, and measurements made from a box operated on this cable. The cable may be suspended from a tree on each side, if trees are available, or from posts, as shown in fig. 1. The height of the posts will depend on the height of the banks and the change in

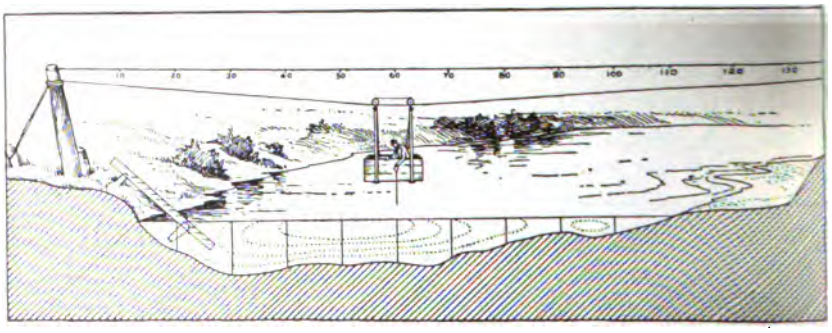

FIG. 1.—Cable station, car, gage, etc.

river stage. The cable should always be so high above the stream that the car will be several feet above water level in the middle of the span when the stream is at flood stages. Each end of the cable, after passing over the posts, is fastened to a timber or heavy iron rail called a "dead man," at least 4 to 6 feet long and 6 to 10 inches in diameter, one end of which is buried 3 or more feet in the ground. Near one end, between the support and the dead man, a turnbuckle should be inserted for tightening the cable when the sag becomes too great for easily moving the car. The equipment for a cable station includes the following items for spans of from 100 to 300 feet: A five-eighths inch galvanized-wire cable; eight Crosby clips, costing about 40 cents each; two 6-inch galvanized-iron pulleys; one turnbuckle (right and left hand thread), with 2-foot capacity, and one gaging car or box 3 by 4 feet by 1 foot deep, made of common lumber and painted. The turnbuckle must generally be made to order, of wrought iron, and will cost from $3 to $5. Above the main cable a wire (preferably common barbed wire) is stretched, to which are attached tin

A. CABLE POST AND CAR.

B. BOAT STATION.

or galvanized-iron tags, marking the intervals at which measurements are made. The tags should be of different shapes—round, oval, or rectangular—with suitable notches to indicate their distance from the initial point, or should have numbers clearly marked upon them, so that no confusion can arise as to the units, tenths, etc.

For spans of less than 100 feet, a one-half inch cable anchored as above described and supported on 10 by 12 timbers set 4 feet in the

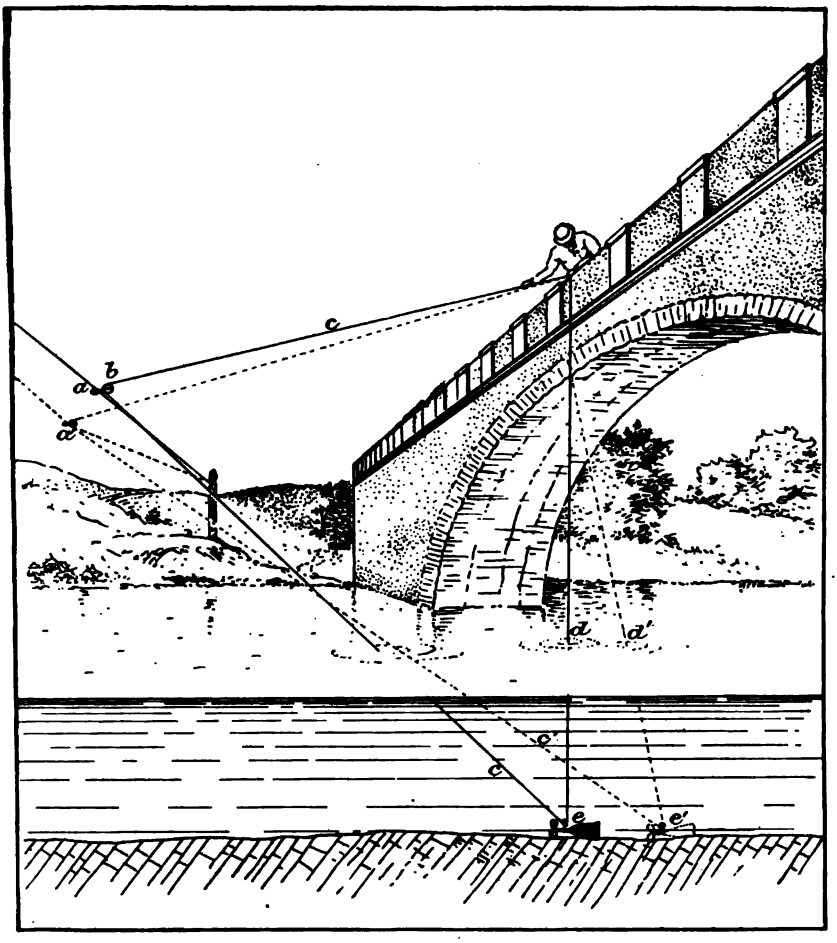

FIG. 2.—Method of manipulating stay line from small cable.

ground will serve. The cable may be attached directly to an iron bolt, 1 inch in diameter, set through the post near its upper end. If two or three bolt holes are made through the post they may be used to aid in adjusting sag in the cable, the bolt being shifted from one hole to another, if necessary. The posts should incline back from the water somewhat, so that they may not be liable to bend under stress.

Boat stations.—Where a bridge is not available, and the width of the

stream is too great for the use of a cable and the depth too great for wading, measurements are made from a boat. For description of measurements by wading see page 21. Two small cables, three-eighths inch in diameter, one to be used as a tag line, the other as a stay line, as shown in Pl. I, *B*, should be used. The meter should be operated from the upstream end of the boat, and should be held several feet away from it.

If the timber that carries the meter and projects from the boat is marked in feet and tenths, it will be found helpful in measuring depths and in lowering the meter to any desired depth. To lower the center of the meter to a depth of 3 feet, first lower it until the center is at the surface, then grasp the line carrying the meter at a foot mark and let the meter descend until the hand has moved over three of the foot marks; the center will then be 3 feet below the surface.

In measuring from a boat two assistants will ordinarily be necessary, one to operate the boat and another the meter while the hydrographer keeps the time and makes the record. A rod marked to feet and tenths is convenient for making soundings from a boat.

GAGES.

Two forms of gages for measuring surface fluctuations are in use—either a timber, vertical or inclined, fastened rigidly to the bank or to some permanent object, as a tree, pile, or bridge pier; or a chain gage attached to some permanent part of a bridge.

A timber gage should not be smaller than 4 by 4 inches and should be marked to feet and tenths of feet vertical depth. Occasionally a tree growing over the water will furnish a good support for a gage; a bridge pier is usually not a good support because of the suction and consequent lowering of the surface, and because it is usually too far from the shore to enable the gage to be easily read.

A vertical gage in two sections may often be used to advantage. The upper part should be in some protected position on the bank for use at high water only; the lower part, for low-water stages, should be of such length that it will be submerged during high stages.

The gage markings should be as permanent as possible. U-shaped galvanized iron staples make good marks. When paint is used the divisions should be indicated by V-shaped grooves one-fourth inch wide and one-eighth inch deep. These grooves should be painted black and the surface of the rod painted white.

In order that minus readings may never be necessary, the "0" of the gage should be 3 feet below lowest known low water, or, in permanent channels, level with the bottom in the deepest place.

It is often possible to use an inclined timber gage where a vertical one can not be used on account of the danger of its being destroyed.

An inclined timber gage should be placed where it is least exposed to drift, and where the water is quiet, so that it can be accurately read; it should have its lower end always under water and be bolted or spiked to posts firmly set in the bank. It should be marked to read to vertical feet and tenths direct, and for this purpose an engineer's level or carpenter's square and level are necessary to determine what distance along the rod corresponds to a foot vertically. It is usually better to

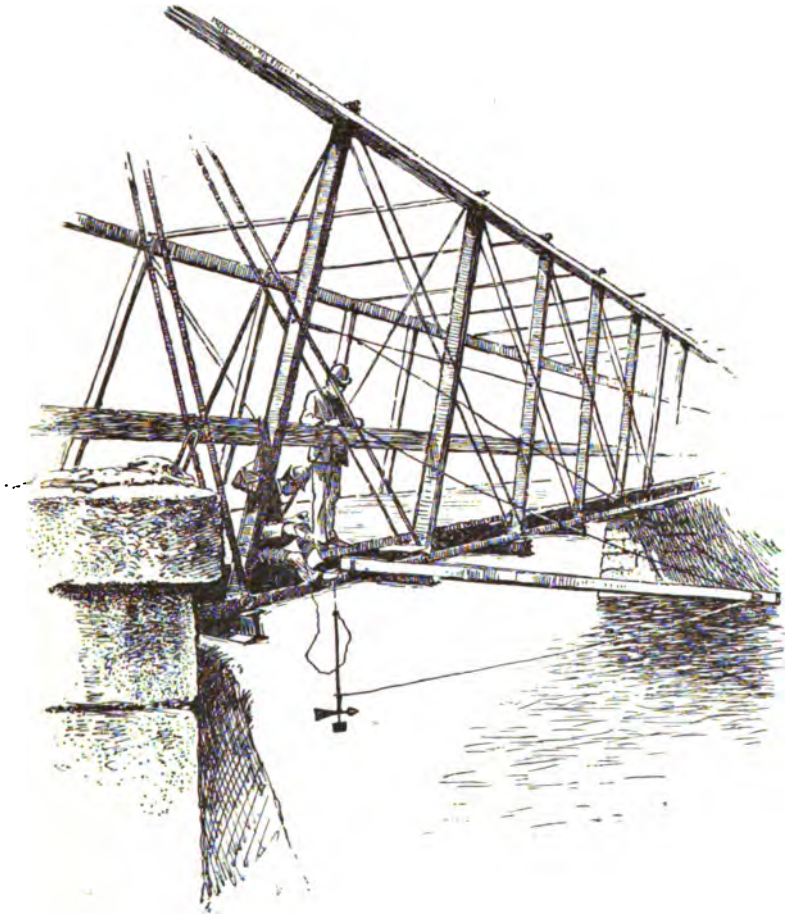

FIG. 3—Method of attaching stay line to meter by use of pole.

put the rod in position, place a few of the foot marks on it, remove the rod and complete the markings, and then permanently fasten it; or the slope of the rod may be taken, and a corresponding scale marked upon a board of the proper width may be afterwards firmly fastened to the gage timber.

The U. S. G. S. standard chain gage, shown in fig. 4, is to be used where ice, logs, and drift will destroy a timber gage, or where, for any

cause, a timber gage can not be used. It must
box with a down-spout to protect the weight, an
locked. The length of the chain will vary som
ence in length must be determined at each visit
and allowed for when a discharge measurement i
the gage without a level there should be a bench
of the bridge, from which the elevation of the
read with a steel tape and weight. On the und
cover should be marked the length of the cha

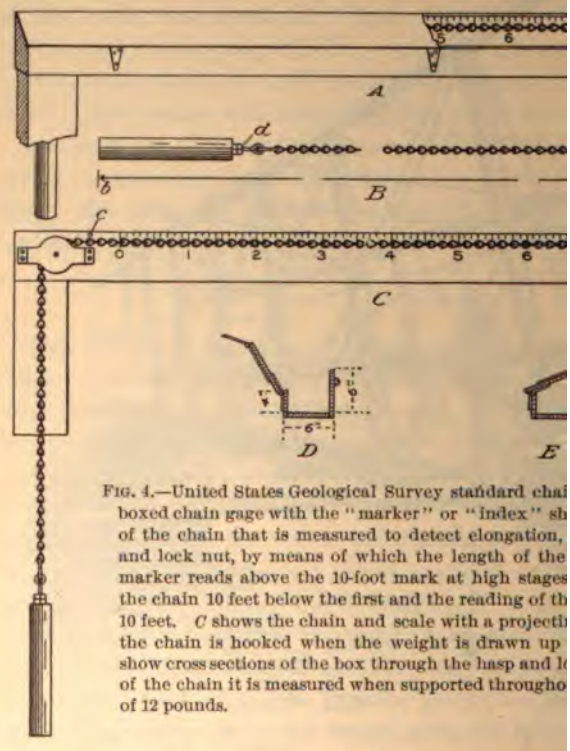

FIG. 4.—United States Geological Survey standard chai
boxed chain gage with the "marker" or "index" sh
of the chain that is measured to detect elongation,
and lock nut, by means of which the length of the
marker reads above the 10-foot mark at high stages
the chain 10 feet below the first and the reading of th
10 feet. *C* shows the chain and scale with a projecti
the chain is hooked when the weight is drawn up
show cross sections of the box through the hasp and l
of the chain it is measured when supported througho
of 12 pounds.

installed and the elevation of the "0" of the
checking the gage will then always be at hand f
ness of the gage reading. For example, suppo
water is 20 feet below the bench mark when th
the chain length is 20.40 feet. At the next vi
feet, and the distance from the water to the ben
the chain has stretched two-hundredths of a foo
the gage pulley has changed. By measuremen
be 20.42 feet in length and should be shortene
foot by the adjusting device at the upper end
bench mark on the bridge should occasionally b

All the hardware for the gage can be obtained at the Washington office, including the lock. It is necessary that the standard United States Geological Survey lock be used, so that the inspector can read the gage when he visits the station.

At stations where the importance of the work calls for a greater degree of accuracy than is given by reading the gage to half-tenths, a low-water gage can be used, marked to the quarters of tenths, and read to the nearest quarter of a tenth, or marked to one-hundredths of a foot, and read to hundredths of a foot, the marking and reading to be determined by the district hydrographer.

A scale 2 or 3 feet in length and marked to hundredths of a foot can be used in some cases to advantage for measuring down to the water's surface from a well-defined point conveniently located.

BENCH MARKS.

There should be at least two bench marks at each gaging station; one preferably a copper plug set in a rock above high water, where it will not be disturbed, the other a point on the ironwork of the bridge from which the elevation of the surface of the water can be read with a steel tape. In sand and alluvial river bottoms where rock can not be found the United States Geological Survey iron post can be used. A surveyor's bench mark is sometimes found on a bridge, and it is therefore necessary to mark the bench mark with "U. S. G. S." so that it can be distinguished from other marks of a similar kind. A cross cut on a ledge of rock or bridge abutment is sometimes used; a spike in a tree is only a temporary mark. Bench marks should be so placed that they can be easily found, and a full description forwarded to the Washington office, with the "description of station."

STAY LINES.

In a swift current the meter will be carried downstream in spite of any weight of lead that can be operated by one man. To keep it directly underneath the point of measurement a stay line is used. This stay line may be a one-fourth-inch steel galvanized cable, fastened to posts on the banks, carrying a small pulley. The meter is operated from this stay line, as shown in fig. 2 (page 13).

A pole projecting upstream from the bridge is sometimes used in place of a stay line for keeping the meter at the proper depth. The meter is operated from this pole, as shown in fig. 3 (page 15). It is, however, difficult, even with these devices, to keep the meter at the desired depth, so that it is usually better to measure the velocity 1 foot below the surface and apply a coefficient to obtain the mean velocity. These devices can be used to advantage in obtaining characteristic vertical velocity curves from which this coefficient may be derived.

MEASUREMENTS OF DEPTH.

Factors for computing discharge.—The volume of water flowing in a stream in one second, or the discharge, is a product of two factors—mean velocity per second and cross section. The cross section is the product of two factors—mean depth and width. If the cross section were a rectangle and the velocity constant in all parts of it the measurement of discharge would be a simple matter, but the cross section of a natural stream is usually irregular in shape, and the velocity varies from the bank to the center and from the bottom to the surface. Considerable judgment is necessary, therefore, in selecting points in the cross section where depth and velocity should be measured, and in determining the method of measuring velocity that will secure a proper degree of accuracy.

It is found most convenient in current-meter work to divide the stream into parts 1, 2, 4, 5, 10, 20 feet in width, depending on the size of the stream and unevenness of the bed and to find the area, the mean velocity, and the discharge through each part separately. The total discharge is the sum of the discharges through the parts.

Fig. 1 (page 12) shows a meter station where measurements are made from a car suspended from a cable. The tags on the tag line directly over the cable mark the points where measurements of depth are made. The inclined gage is shown on the left side. The cross section is shown divided in parts by vertical lines directly over the tags. The dotted curved lines in the cross section are lines of equal velocity. The figure shows velocity being measured under one of the tags.

Soundings.—Soundings should be taken at intervals across the stream, sufficiently near together to enable the cross section to be computed to the required degree of accuracy. The distance between the soundings should depend upon the size of the streams, evenness of the bed, and the degree of accuracy required. For small streams, the interval may vary from 2 to 4 feet; for streams of moderate size, from 5 to 10 feet; and for larger streams from 10 to 20 feet, except around obstructions, where they should be taken about 5 feet apart.

A small, round weight is not so good for sounding as a large, flat one, because if the bed is soft or rough the former will settle into the bed or between the projections and give too great a depth.

When a very heavy weight is used for sounding in swift water it is well to have foot and half foot marks on the sounding line, so that the depth can be read when the lead rests on the bottom. Different colored bits of ribbon firmly tied to the sounding line and wrapped with insulating tape will generally answer the purpose. The part of the line that is immersed may preferably be of picture wire.

On account of the great difficulty of obtaining accurate soundings at high stages of water, depths for flood measurement should be com-

puted from soundings taken at a lower stage, either just before or just after the flood, provided the channel is of a permanent character.

The soundings can be taken with the meter on the line when the velocity in a vertical is being measured if the velocity is less than 3 feet per second. Care must be taken, however, to lower the meter gradually and not allow it to suddenly strike the bottom.

In sounding in a swift current the lead should be directly underneath the point of observation and the line should not be bowed downstream. When the bed is very uneven two or more measurements of the depth at each point should be taken and the mean used. This can easily be done by holding in one hand the point on the line which touches some well-defined point on the bridge when the lead rests on the bottom, and comparing this point with the corresponding points on the line obtained by two or more trials. The distance measured on the line from the point obtained as the mean of these two or three trials to the point on the line when the lead is drawn up so as to just touch the surface of the water is the mean depth. When more than 40 pounds of lead are used on the meter a cotton rope should be used for handling it, as the cable is not strong enough.

The initial point for soundings should be so marked that it can easily be recognized, and the points at which soundings are made should be clearly marked on the bridge or on the tag line of the cable or boat station.

MEASUREMENTS OF VELOCITY.

General statement.—Velocity should be measured in each vertical where a sounding is taken except where the change is small, when it should be measured in alternate verticals. Several methods are in use for obtaining mean velocity in a vertical. They may be classified as single point, multiple point, and integration. In velocity observations the revolutions of the meter wheels should be counted for two equal periods so as to check the count. These periods are usually 50 seconds each.

Single-point method.—Three single-point methods are in general use. In one, usually called the 0.6-depth method, the meter is held at the depth of the thread of mean velocity; in another, called the flood method, the meter is held 1 foot below the surface; in the third the meter is held at mid depth. In each of these methods it is necessary to apply a coefficient to reduce observed velocities to mean velocities. The advantage of the first one is that it is rapid and simple and the coefficient is unity. The advantage of the second is that it is the only method that can be used during a flood. The third method is seldom used.

The mean-velocity method, ordinarily called the 0.6-depth method, will give very good results where the conditions are good—that is,

where there is a nearly straight channel with little obstruction, a bed regular in shape, and no sudden changes in velocity. The thread of mean velocity for such condition varies from 0.55 to 0.65 of the depth, its position depending on the depth, the ratio of width to depth, and the roughness of the bed. For broad, shallow streams with gravelly beds, of depth from 1 to 3 feet, holding the center of the meter 0.57 depth below the surface will give satisfactory results. For ordinary streams, of depth from 1 to 6 feet, holding the center of the meter at 0.6 depth below the surface will give satisfactory results.

The flood method is to be used in making measurements at very high stages, when the single point and integration methods can not be used. The meter should be held 1 foot below the surface and a coefficient applied to the measured velocity to reduce it to mean velocity. The value of this coefficient varies from .85 to .90. An easily recognized mark on the meter line, one foot above the center of the meter, will be found useful in keeping the meter at the proper depth. (See also flood measurements, p. 23.)

Integration method.—In the integration method the meter is kept in motion either from the surface to the bottom and back again to the surface in a vertical line, or diagonally from the surface to the bottom and back again to the surface, while it is at the same time moved across the channel. The latter, called the zigzag method, is seldom used, except in comparatively small artificial channels.

The vertical-integration method, consisting of moving the meter from the surface to the bed and back again to the surface, counting the number of revolutions and noting the time, gives satisfactory results if the meter is moved slowly and at a uniform speed. It is a better method than the others where time is limited and where the conditions are poor (crooked channel, obstructions, etc.); also where the surface is retarded by drift, logs, or ice. It is a very good method for checking results obtained by the single-point method. This method is more difficult for one man than the point methods, and gives somewhat less information.

Multiple-point methods.—These consist of top and bottom; top, mid depth, and bottom; and vertical velocity curve. In the top-and-bottom method mean velocity is taken as the half sum of the top and bottom velocities. In the top, mid-depth, and bottom method the mean velocity is taken as one-fourth the sum of the top and bottom velocity and twice the mid-depth velocity. $V=\frac{1}{4}(T+B+2M)$. In the vertical-velocity-curve method the mean velocity is computed from the velocities observed at several points in each vertical, as shown on pages 50–51.

The top-and-bottom method does not give satisfactory results where the bed is uneven. The results are, as a rule, too small. In a very shallow stream, 3 to 12 inches in depth, with sandy or fine gravel bed, satisfactory results are obtained by this method if the center of the meter is held 0.15 of a foot below the surface and the same distance

A. CURRENT METER RATING STATION AT DENVER, COLO.

B. METHOD OF MAKING DISCHARGE MEASUREMENT BY WADING.

above the bed. If the bed is coarse gravel (particles 1 to 2½ inches in diameter) the center of the meter should be held 0.15 of a foot below the surface and from 0.3 to 0.4 of a foot above the bed.

The vertical-velocity-curve method should be used when there is abundant time, for it gives more accurate results than either of the other methods, but it requires so much time that it is seldom employed except to check results obtained by one of the other methods. From three to eight observations are necessary in each vertical, each requiring from 50 to 100 seconds of time. A few vertical-velocity curves should be obtained at certain selected points in the cross section if possible.

Low velocity limitations.—The current-meter and weir discharge comparisons made at the hydraulic laboratory of Cornell University, described in Water Supply Paper No. 64, show that current-meter measurements of velocities less than about 0.4 or 0.5 foot per second are not reliable. The meter discharge is less than that shown by the weir, the error increasing as the velocity decreases and as the friction of the meter increases. For these reasons it is not advisable to attempt the measurement of the discharge at a place where the mean velocity is less than about half a foot per second. Inasmuch as there is always a small area of low velocity near the banks and around piers, a rule was made by the hydrographer in charge, March, 1903, that hereafter when the velocity at a station becomes less than half a foot per second in more than 15 per cent of the cross section, the measurements there should be discontinued. At many stations where current-meter measurements can not be made when the flow becomes sluggish at low stages, a place can be found within half a mile of the station where the velocity can be measured by wading.

At each permanent gaging station where a high degree of accuracy is required a somewhat extended study should be made at different stages by the vertical-velocity-curve method to test the reliability of the results obtained and to derive coefficients applicable to other stations having somewhat similar conditions.

Measuring discharge by wading (*see Pl. II, B*).—Discharge can often be measured more accurately than at the gaging station by wading in some chosen section where the conditions are good. Iron rods three-eighths inch in diameter and at least 3 feet long, with a long slot or thumbscrew in one end and the other end pointed will be found convenient for holding the ends of the tape during the measurements. Depth can be measured with a light rod, marked to feet and tenths, that can be made in a few minutes. The hydrographer should hold the meter as far to the side of him as possible and a little upstream so that there will be but slight obstruction offered by his body (see Pl. II, *B*). In a very small stream measurements should be made from a plank laid across it instead of by wading.

CHECKS.

The field work of the discharge measurement should not be considered complete until it has been checked. The chain gage can be checked with a steel tape, as described on page 16. The observations of velocity can be checked rapidly by the integration method, and the computed discharge partially checked by plotting it on squared paper and comparing with station rating curve of the preceding year. If any discharge varies from the station rating curve by more than 5 per cent for ordinary stages at a station where the conditions are fair to good, or by more than 8 per cent where the conditions are fair to poor, the hydrographer should seek the cause in change in channel or in mistakes.

CLASSES OF DISCHARGE MEASUREMENTS.

Minimum-flow measurements.—Records of the minimum flow of a stream are in nearly all cases very important, and special effort should be made to secure them every season at each important station. It is not very important to determine the smallest amount that flows past the station during the season, for this minimum may occur when the greater part of the natural flow is being held back by dams; what is desired is the average flow for the month or week when the flow is least.

A larger number of discharge measurements are necessary to define the lower part of the station rating curve than any other part, because small changes in gage height have a much larger proportional effect on the discharge for the lower stages than the higher ones, and because the slope changes faster in the lower than in the higher parts of the curve.

When there are daily fluctuations in the discharge—as, for instance, where the water is held back by dams—care should be taken to have the gage read at such times that the reported "daily gage height" is the mean for the day; for example, if a gage is below a dam that holds back the water during the night, one reading should be taken when the water wheels are in use and one when they are shut down.

It is often advisable to have a low-water gage in addition to the one for other stages, one that can be read by the observer easily and with a greater degree of accuracy.

Facts in regard to the minimum flow of tributaries of each stream and their suitability for power and water-supply purposes should be collected by the hydrographer and reported to the Washington office, on form 9–213 or by brief reports. It is well to give also the dates and amounts of precipitation in inches at the nearest Weather Bureau station for some days preceding the date of measurement. Such facts can usually be obtained at small expense, and they add greatly to the value of the discharge records.

Flood-flow measurements.—The stage of a stream during flood usually changes so rapidly that a discharge measurement made at such a time, to be of greatest value, should be made in less than two hours; three or four observations of the velocity 1 foot below the water surface between each pier and one or two between each pile pier will usually answer this purpose. Depths can be obtained from previous soundings at a lower stage or from the cross section of the river at the station, developed after the flood.

Care must be taken to protect the current meter from injury by drift.

When for any reason the meter can not be used, the surface velocity can be obtained by means of floats.

In wide streams where the conditions between piers are similar, if the velocity is 5 or more feet per second and the bridge spans are 150 feet or more in length, it is not wise to attempt to measure velocity nearer to a pier than about 25 feet. When the measurement is taken the area of the pier should be neglected in computing the discharge. If the velocity is less than 5 feet per second the area of the pier (or piles in the case of trestle work) should be subtracted from the area in computing the discharge.

The gage height at which overflow of banks takes place should be noted, also any backwater effect. Facts in regard to character and extent of damage done by the flood should be obtained; also effect of obstructions upon the height of the flood.

Distribution of discharge measurements.—As far as possible discharge measurements should be made at such river stages as will give a point on an undefined part of the station rating curve. Frequently there are several measurements made at about the same gage height and no measurement for 2 or 3 feet stage above or below them. By instructing observers to telegraph when the river reaches a desired stage the hydrographer can time his visit so as to obtain a point on the curve at which no measurements have been made.

Very often the hydrographer can obtain two or more points on the rating curve at a single visit. By remaining a couple of days at the station when the stage is high he can make four or more discharge measurements cheaply, which may serve to fix a considerable part of the curve.

Miscellaneous discharge measurements.—A miscellaneous discharge measurement is one that is made at a distance from a permanent or temporary gaging station. The place of measurement should be referred to some easily found landmark—as, for example, "500 feet upstream from county bridge, 3 miles northwest of ———," and the elevations of the water surface should be referred to some point that can be easily found and again used—as, for example, "10 feet below upper surface of floor beam, first span north end Southern Railway bridge, 3 miles south of ———." Facts in regard to the behavior of the

stream, its minimum and maximum flow, and a comparison between the discharge at the time of measurement and low flow should be ascertained and reported.

When measurements are made at several places along a stream during a reconnaissance, allowance should be made for rain that has fallen in the interval between measurements between the places and on tributaries entering the stream between points of measurement.

Winter discharge measurements.—The winter discharge of the important streams is desired at permanent gaging stations, where the conditions are such that reliable data can be obtained at reasonable cost. If ice does not interfere with the work, it should be continued during the winter, as at other seasons of the year.

If anchor ice forms at a station, a record should be kept showing the date of its formation, the height of backwater due to it, and whether much or little water is flowing at the time. Facts in regard to the rise in the stream that lifts the ice but does not clear the channel should be noted.

At stations where solid ice forms, the observer should visit the station at least once a week to read the gage and note the condition of the stream. If the gage is a chain one, the ice should be cut away around the gage, the gage read, and the thickness of the ice measured, also the distance of the surface of the water above or below the surface of the ice. The observer should also note whether the ice is rough or smooth on the under side and the distance to open water above and below the station.

A rod with a crosspiece on the lower end, forming a T, is convenient for measuring the thickness of ice.

The vertical-integration method of measuring the velocity (by moving the meter slowly from the surface to the bed and back to the surface again, counting the revolutions, and noting the time) will be found satisfactory under ice. Some vertical velocity curves, however, should be taken at each measurement. A special station rating curve must be used for periods when ice interferes with the natural flow.

GAGE READINGS.

Computations of discharge and run-off are usually based on gage readings taken one or more times daily. If any of these are in error, the results obtained from them are in error also. Every effort should therefore be made by the hydrographer to secure thoroughly reliable gage readers. No pains should be spared to teach them to read the gage correctly and to properly record and report the readings. They should realize the importance of the records they are taking, and should know that means are being taken to see that the records are reasonably correct.

At each visit to the station the hydrographer should examine the gage reader's book and make comments thereon. The reading of the observer and hydrographer on the day of visit should be compared.

A gage reader is more likely to read with regularity and accuracy a gage that can easily be reached and seen than one which requires unusual effort or risk to read; hence, except in rare cases, a chain gage should not be placed on a high railway trestle bridge, nor on a structure where it is necessary for the observer to climb, nor where he is obliged to kneel down and reach out over some part of the structure.

Rod gages frequently become waterworn and covered with dirt, so that they are difficult to read. Occasionally the bed fills in around the lower end of the gage, so that it is necessary to keep open a channel of running water to the gage. These points and many others the hydrographer must keep in mind and provide for in order to secure satisfactory records of daily gage heights.

STANDARD CROSS SECTION.

There should be prepared for each permanent gaging station a cross section of the stream showing the contour of the channel to points on each bank above the highest flood water, the piers, and other obstructions, and showing elevations referred to the zero of the gage. An engineer's level or plane table will be necessary for this purpose. From such a cross section approximate depths can be found for any gage height; also changes in channel due to scour or fill. It also assists very materially in the preparation of the station rating table.

DATA ON FLOODS.

Hereafter there will be prepared at the end of each year a water-supply paper on the destructive floods of the year, showing their magnitude and extent, the destruction wrought by them, and the engineering features involved in the prevention of their destructive action. When a notable flood occurs in the area in charge of any district hydrographer he should make a special effort to visit the locality during the flood, or very soon thereafter, and obtain all facts possible concerning the height, quantity of water, destruction wrought, and reasons therefor, and prepare a report thereon, adding comments concerning the action of bridges, buildings, levees, etc., along the stream. In such investigations care should be taken to verify data not obtained from actual observation, especially any data necessary for computing discharges. If discharge is computed from data obtained principally from flood marks, special care must be taken, because these marks are often misleading.

RECONNAISSANCE.

A reconnaissance of a stream is made for the purpose of locating a gaging station, investigating water power or water storage possibilities, or studying the destruction wrought by floods or the pollution of the water.

All data that have any bearing on the question studied should be collected, and sketches should be freely used, showing the relative positions of objects described. The notes should be very full and clear, so that they will convey correct ideas of facts after they have been in part or wholly forgotten.

A reconnaissance for the selection of a gaging station or for investigating power and storage possibilities is usually made when the stream is low, as data collected for these purposes are more valuable at this stage than at the high stages.

The instruments used are a hand level, compass, steel tape, and current meter.

The topographic features of the watershed, such as the elevation, slopes of surface, width of valleys, character of rock and soil and vegetation, should be noted; the slope of the stream, location of the used and unused power, location and magnitude of principal tributaries, high-water marks, the extent to which the water is used for industrial purposes, kind of industries, and the kind and sources of pollution should also be noted.

The discharge of a stream and its principal tributaries should be measured, and a temporary bench mark should be left at each point of measurement, so that if a subsequent measurement is made at that point it can be compared with the former.

DESCRIPTION AND CARE OF INSTRUMENTS.

GENERAL STATEMENT.

Hydrographers are responsible for the care of current meters and other Government instruments intrusted to them, and are required to account at stated times for all such property in their keeping. Since the accuracy of stream gagings depends largely upon the correct working of the current meter, great care should be exercised in handling these instruments and in keeping them properly adjusted. Inaccurate work and poor results can often be traced directly to neglect in caring for instruments.

CURRENT METER.

The following description and suggestions regarding the use and care of the small Price electric current meter (see Pl. III) are intended for the guidance of hydrographers in the field:

When in use the meter is suspended by a double conductor cable of No. 14 or No. 16 flexible copper wire, heavily insulated. Wire of that

PRICE ELECTRIC CURRENT METERS, WITH BUZZERS.

size is of sufficient strength to hold the meter and weights, and it obviates the necessity of additional rope for suspending the equipment when the weight used is less than 40 pounds. The cable is attached to the meter with a spring snap hooked into the circular end of the trunnion P, fig. 5. The heavy copper wires are connected with the meter binding posts *h* and *d* by smaller and more flexible wires. The wires connected with the binding post *d* should be threaded through the metal loops on the yoke *o*, within the trunnion frame, and at *g*. It is desirable that these wires should be flexible and loose, to allow the meter to swing free in the vertical plane when it is in use.

Lead weights (*a*, fig. 6), provided for the purpose of holding the meter steady in moving water (the higher the velocity of the stream

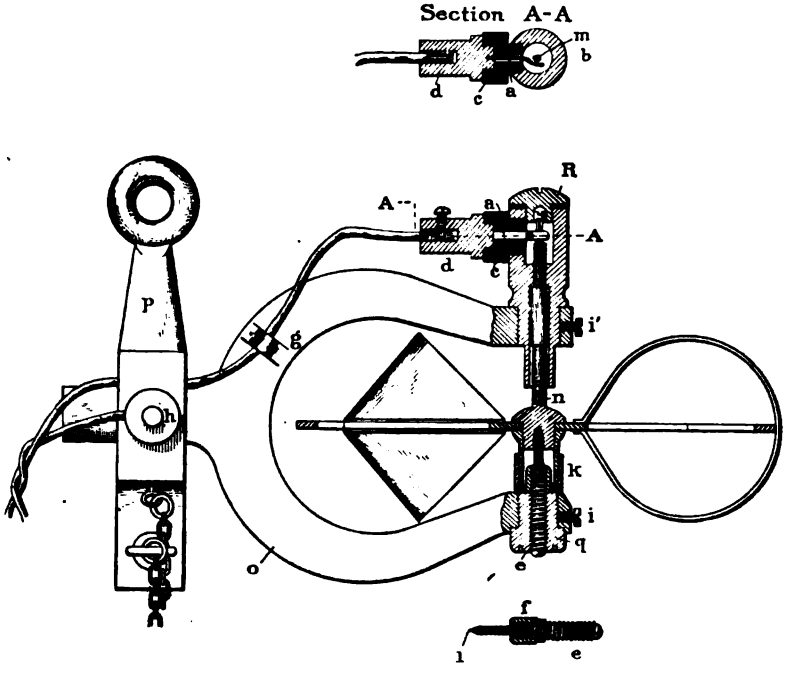

FIG. 5.—Cross section of small Price electric current meter, showing details.

the greater the weight), are attached to the lower end of the trunnion by means of a detachable weight stem (*b*, fig. 6).

The weight vane (*c*, fig. 6) should be attached to the weights at all times when the meter is used suspended from a cable. When gaging small or shallow streams it may be necessary to make observations by wading. Under such circumstances it will be more convenient to dispense with the lead weights and attach the meter to a light rod or pole. If desired a brass standard can be supplied for attaching the meter to the rod.

The meter is supported in a trunnion or hanger (P, fig. 5), and is free to swing in a vertical plane. One revolution of the wheel or cups is

indicated by a buzz of the electric buzzer, the observer being required to count the number for a certain interval of time, preferably fifty seconds, as the computations can more easily be made from that number. A second observation of the same length of time should immediately follow the first observation in order to verify the count.

The vertical axis (*n*, fig. 5) to which the cups of the current-meter wheel are attached, and hereinafter referred to as the cup shaft, terminates at the lower end in an inverted cone, which bears or turns on the cone-shaped point of *e*, fig. 5. The part marked l, fig. 5, will be referred to as the point bearing. This is the most delicate part of the meter, and should be treated with the greatest care, as it is made of highly tempered steel and is liable to be fractured or broken. To protect this sharp point bearing while the meter is being shipped or carried, a milled sleeve (*k*, fig. 5) is provided. This sleeve is threaded on the inside, and screws up or down on the screw thread on the extension of the lower end of the cup shaft shown in fig. 5. When the meter is not actually in use, and before it is put into the wooden

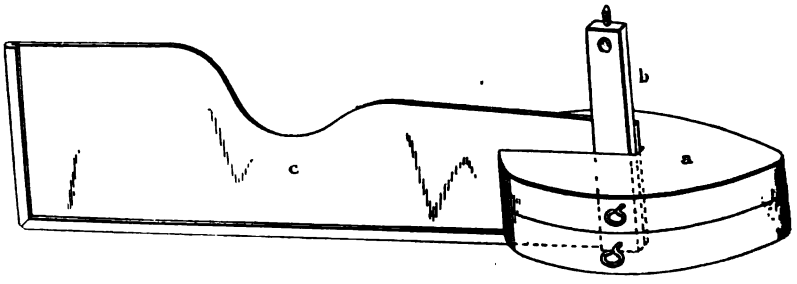

FIG. 6.—Weights and weight vane of small Price electric current meter.

case after using, the milled sleeve *k* should be screwed down until it bears on the top of *q* and raises the cups and cup shaft off the point bearing, thus protecting it from possible injury. This sleeve should not be screwed down very tight, as the shoulder of the journal on the upper end of the cup shaft will be thrust hard against the end of its bearing and perhaps be injured.

When preparing to make a measurement with the meter the milled sleeve should be screwed up on the cup shaft far enough to make it absolutely certain that the cups are turning on the point bearing and are working with the least possible friction. This sleeve is milled for adjustment with the fingers and not with a wrench or pliers.

If the point bearing becomes injured and it is necessary to replace it, an extra one will be found in the meter case. To replace the bearing slacken the screw *i*, fig. 5, with the small spanner wrench provided with the meter, remove the large nut *q* with the point bearing locked in it. After its removal hold the head of the nut *q* firmly with a small wrench or pair of pliers, slip the small spanner wrench over

the flat sides of the nut *f*, and loosen it. Remove the nut *f* with the fingers, and with a small screw-driver take out the damaged bearing from the large nut *q*. Replace the large nut *q*, and with the spanner wrench screw it firmly into yoke *o*. Remove the lock nut from the point bearing to be inserted, and with a screw-driver send the latter through the large nut *q*. Great care should be exercised in making this most important adjustment of the current meter. A slight amount of play or movement of the cups up and down must be allowed, so that they will revolve freely and without friction. When the point is almost up turn it, a part of a revolution at a time, with the screw-driver, until the proper adjustment is obtained. After the adjustment has been made a full turn of the point bearing in the nut *q* would crush the sharp point of the bearing *e* into the inverted cone of the cup shaft and break the point. After the point has been satisfactorily adjusted remove the nut *q*, with the point in it, put the lock nut on the point, and draw it down on the face of the large nut and thus preserve the adjustment. Should the adjustment of the meter be too tight the results obtained from its use will be erroneous. An examination of the meter when received from the Survey will give an idea of the proper adjustment. Do not disturb the adjustment of the meter unless it is necessary to put in a new point bearing.

Section A–A, fig. 5, shows the construction of the binding post and contact spring of the meter. A flexible, well-insulated copper wire (No. 20, or smaller), is drawn through the metal loop *g* of the yoke *o*, and the end of the wire free from insulation is thrust into the metal binding post *d* and secured by the small set screw. This metal binding post terminates in a slender platinum spring (*a*), which extends through the hard rubber nipple *c* into the contact chamber to *m*. The top or upper end of the cup shaft terminates in the contact chamber with an oval-shaped eccentric (*m*), which makes a contact with the spring *a* at each revolution of the meter cups. This contact can be prolonged or shortened by bending the point of the contact spring in the contact chamber in the desired direction. The oval-shaped eccentric end of the cup shaft is detachable, and can be removed by taking off the cap R, holding the cups firmly, and applying a screw-driver to the small slotted head.

The insulating nipple *c* is made of hard rubber, and is likely to break if the binding post *d* receives a sharp blow. An extra insulating nippple will be found in the small tin case in the meter box. Before removing or replacing the binding post *d*, take out the eccentric-shaped top of the cup shaft *m*. If this is neglected the contact spring will be destroyed by the turning of the binding post.

In using and handling the meter care should be taken that it does not fall or get a hard knock which may injure the cups or cup shaft.

An injury to either will change the rating of the meter, and if this occurs it should be immediately returned to headquarters for repairs. When using the meter in streams containing grass or moss, examine the instrument frequently to see that nothing has lodged on or wrapped around the cup shaft *n*.

If the meter has not been used for some time before a measurement is made it should be carefully cleaned and oiled.

BATTERY AND BUZZER.

In charging the battery cell used with the electric buzzer furnished with the current meter, one-half teaspoonful of bisulphate of mercury is sufficient. Fill the cell with water and insert the zinc pole with the rubber stopper attached. When putting the battery cell in the leather case be sure that the small platinum point on the lower end of the cell and the screw head of the rubber stopper make perfect contact with the copper plates. If the buzzer makes but a faint clicking sound instead of a buzz, remove the metal cap covering the buzzer, and with a knife blade adjust the small upright brass point by bending until the armature produces the desired sound. Never allow the liquid to remain in the battery cell over night, as it will generate gas and produce pressure sufficient to cause the cell to leak at the rubber stopper, and the solution escaping will destroy the leather of the brass parts of the buzzer. When measurements are needed requiring a long time for completion the stopper should be removed occasionally to relieve the pressure due to the gas.

When preparing to make a discharge measurement charge the battery cell of the buzzer, remove the meter from the case, take out the weights, weigh stem and vane, combine them as in fig. 6, and attach them to the meter with the spring pin provided; turn up the milled sleeve *k*; be sure that the meter is well oiled, and turn the cups to see that they revolve freely and that the buzzer responds to each revolution.

After making a discharge measurement, or at the close of a day's work with the meter, unscrew the large nut *q*, with the point bearing in it, and examine the point carefully to detect injury. With a piece of soft cloth remove the water in the opening at the lower end of the cup shaft, oil generously, and replace the bearing. Then remove the screw cap R, turn the meter over to allow the water to run out of the contact chamber, put in a plentiful supply of oil, and replace the cap; this protects the top of journal bearing of the cup shaft from rusting. After this has been done turn down the milled sleeve *k*, to protect the point bearing while carrying the meter, remove the battery cell, empty out the liquid, wash the cell with water, and pack the meter carefully in the case, when it will be ready for future use.

RERATING OF METERS.

Whenever a meter point is found to be broken or injured it should be replaced by the other point bearing which accompanies the meter. If the cups are injured or the cup shaft is bent the meter should at once be sent to headquarters for repairs. Every meter that has been used to any considerable extent after rating should be rerated before the beginning of the next field season.

For description of meter-rating stations and method of rating meters see Water-Supply and Irrigation Paper No. 56, page 34.

The rating table of a small Price meter when held with a rod differs somewhat from that applicable to a similar meter when held with a cable and free to tip, so that if this meter is used with a rod care must be taken to employ the proper rating table.

RECORDS AND REPORTS.

General statement.—The regulations which govern all work done by the United States Geological Survey require that all original records of data collected shall be filed for public reference at the office of the Survey, in Washington. These records should be so clear and full in all respects that they will be intelligible at any time to engineers and other persons who may desire to consult them.

Duties of district hydrographers and engineers.—In order to systematize stream measurements the country has been divided into districts, each of which has been placed under the supervision of a district hydrographer or engineer. It is the duty of this officer to superintend all stream measurements in his territory, and he is held responsible to the chief engineer for the character of the work done by the men under him. He should have immediate supervision of all the more important features of the work which can not be intrusted to his assistants. The stream-gaging work, which is carried on in connection with the investigation of irrigation projects, should be under the direct charge of a man designated by the district engineer, through whom he shall report. This man should be held responsible for the proper execution of all stream-measurement work, and all data concerning such work must be transmitted through him.

Care in keeping records.—As the work of the Government is for the general public, it is subject to more severe criticism than the work of private engineers. It is therefore essential that great care be taken in the work and that every possible excuse for criticism be eliminated. All necessary notes should be taken in the field, and, as a special precaution, at the end of each day's work notebooks should be reviewed and amplified while the facts and conditions are still fresh in the mind of the hydrographer or engineer. Careful cross references should be

made, and sketches to show the relative location and details of special features should be freely used.

Computing field notes.—Original computation of all field notes should be made, when possible, in the field, or the notes should be computed at as early a date as possible, and in no case should the hydrographer allow large quantities of undigested data to accumulate.

Checking of records.—All records and notebooks should be examined and checked before they are transmitted to the Washington office. In order to fix the responsibility for errors each book or sheet should be stamped and initialed "Computed by ———," "Computations checked by ———," or "Examined by ———." All records are carefully examined at the Washington office, and if incomplete they will be returned to the district engineer or hydrographer for correction and amplification.

Duplicate records.—The district hydrographer may cause to be prepared such duplicate copies of original notes as may be necessary for reference in his office. Too much duplication should be avoided. The original records may be loaned at any time from the files of the Washington office.

Transmission of data to Washington office.—All data, correspondence, and other material from each hydrographic district should be transmitted through the district hydrographer or engineer to the Washington office. This material should be initialed by the district hydrographer or by some one designated by him. With the exception of single gage-height and discharge-measurement cards, mailed as postal cards, all data sent should be accompanied by a letter stating the amount and character of material furnished. No data will be accepted by the Washington office which is not O. K.'d by the district hydrographer or by some one designated by him.

Standard forms.—As far as possible the standard forms furnished by the Survey should be used in transmitting data. Such information as can not be written on these forms and cards may be transmitted on the separate sheets of paper furnished by the Survey. These sheets should be of the standard letter size, 8 by 10 inches, as this is the size for which the file boxes of the Washington office are designed. Pieces of paper of odd sizes should not be used either for drawings or reports. Sketches and maps accompanying reports should, whenever possible, either be on paper of standard size, or should be folded to conform with this size in order that they may be filed with the written reports.

The following is a list of the forms which have been adopted for general use in hydrographic work. These are intended both to minimize and simplify the work. Whenever forms become too numerous they lose their usefulness. Every effort has, therefore, been made to

reduce the number of forms to a minimum and to prepare these in the simplest manner possible.

List of standard forms.

GAGE-HEIGHT FORMS.

Form 9–176.—Gage-height cards, for transmitting observer's daily gage readings to the resident and district hydrographer, to be forwarded to the Washington office.

Form 9–175.—Observer's gage-height books, for recording daily gage-heights at river stations.

Form 9–212.—Daily gage-height form, for tabulating daily gage-heights, arranged by months, with sufficient space for one year's records.

DISCHARGE-MEASUREMENT FORMS.

Form 9–198.—Current-meter notebooks for recording notes of discharge measurements, vertical-velocity curves, and other meter work.

Form 9–221.—Card for reporting discharge measurements to the Washington office.

Form 9–207.—For tabulating discharge measurements for regular stations, with spaces for all necessary data.

Form 9–223.—For tabulating miscellaneous discharge measurements.

Form 9–244.—For describing the method of and for reporting miscellaneous discharge measurements made by floats, to be used in regular work.

COMPUTATION FORMS.

Form 9–192.—For tabulating daily gage heights and their corresponding discharges, with spaces for totals, monthly means, and run-offs. The form provides for one year's records, arranged by months.

Form 9–210.—For tabulating station rating tables.

Form 9–206.—For tabulating meter rating tables.

REPORT FORMS.

Form 9–197.—Description of river stations; to be used for describing both temporary and permanent stations.

Form 9–237.—For reporting the conditions at river stations and giving the cost of maintenance.

Form 9–213.—For reporting changes and other information in regard to river stations.

Form 9–171.—Monthly report on river stations.

Form 9–245.—Monthly report of services, giving daily employment.

VOUCHERS AND MISCELLANEOUS FORMS.

Form 9–230.—A blank form ruled for miscellaneous tabulation.

Form 9–904.—Cross section notebook.

Form 9–903.—Level notebook.

Form 9–253.—Index blanks for notebooks.

Form 9–019a.—Subvouchers.

Form 9–009.—Service vouchers for field men.

Form 9–005.—Field vouchers.

Form 9–015.—Traveling expense vouchers.

Form 9–918.—Expense account notebook.

Aside from these forms, which are in general use, there are several forms which are used in the Washington office and in special work,

Hydrographers desiring forms for any purpose will kindly forward a description of what they want, and they will, as far as possible, be supplied.

INSTRUCTIONS FOR USE OF FORMS.

Observers' gage-height books (form 9–175).—In the observers' gage-height books are recorded the daily stage of rivers, together with miscellaneous information, such as data in regard to floods, ice, weather conditions, etc. The observer should be instructed and trained to keep the books in as legible condition as possible, and to insure this the hydrographer should inspect the gage-height book at each visit to the station. At the end of each year the observer should be furnished with a new book and the old one should be returned to the district hydrographer's office, where it should be examined, checked, and forwarded to the Washington office for filing.

Gage-height cards (form 9–176).—Gage-height cards should be made out at the end of each week by the local observer and mailed to the district hydrographer's office, where they should be carefully examined. In this examination all missing data, such as the date, name of station, river, State, etc., should be supplied. The gage heights should be compared with the gage heights for previous weeks to see that no unaccountable changes due to error in reading the gage have been reported, and the column for means should be filled in. A copy of the data on the cards should be made for the office of the district hydrographer and the originals should then be forwarded to the Washington office.

Current-meter notebooks (form 9–198).—In these books are recorded the field notes of discharge measurements and vertical velocity curves. In all cases the original data should be written in the book at the time of the measurement. In no case should it be copied from rough notes in other books. Explanatory notes, cross references, and sketches should be added at the end of each day's work. The blank spaces at the head of each page are to be filled in; also the columns for the original data. In the column headed "Depth of observation" the exact depth for each observation should be recorded; "at 0.6 depth" is not sufficient; the position of the initial point, edges of water, edges of dead water, and number of channels should be stated; also the direction and force of the wind, roughness of the bed, and any other factors on which the accuracy of the work depends. A statement of the reliability of the results should be made in the notes. If the measurement is not made at the station the approximate distance and a brief statement of the conditions at the places of measurement should be made. The velocity at the water's edge and at the edge of piers, shoals, etc., should be estimated and recorded. The

cross-section pages at the end of the book are to be used for sketches and additional explanatory notes. Cross-reference notes should be made referring to these pages. Page 48 shows a sample page of current-meter notes with the computations for the same.

Discharge measurement cards (form 9–221).—These are to be made out by the hydrographer and sent to the local office when the measurements are computed. Care should be taken to fill out all portions of the card. Special attention is called to the remarks and the portion for "gage verified and found" (see p. 16 for explanation). Measurement is not complete without this data. Either "At permanent station," "Temporary station," or "Miscellaneous measurements" should be crossed out, so that the card shows at once to which class of measurements it belongs. All cards should be carefully examined before they are sent to the Washington office.

Form 9–213.—In order that the office may keep a complete and systematic record of all the changes and additions at river stations, such as new gages, bench marks, changes in the river bed, equipment, etc., form 9–213 has been prepared. This form is to be used in sending in to the office any data in regard to the regular river stations which affect the measurements or the station in any way. These forms are filed with the regular data for the station, and are necessary in order that the Washington office may thoroughly understand the conditions under which the station has been maintained and the measurements made.

Description of river stations (form 9–197).—Special care should be taken in filling out the description of stations. The description should be so clear that with it a stranger could go to the locality and be able to continue the work at the station. The name of the station should be carefully chosen, and it should definitely locate the station with respect to some of the prominent features in the vicinity, as above or below some creek or tributary, at a certain bridge, near a certain township line, etc. If there has ever been another station on the same stream in the vicinity of the new station, the relative location of these two stations should be stated, and, if possible, the relation of the zeros of the two gages. In order to facilitate the preparation of rating curves it is desirable that care be taken in preparing a sketch of the cross section of the stream. This, with the other sketches, may be made on cross-section paper and attached to the description. It is also desirable to cut from a topographic sheet or some other map a section which will show the location of the station.

Indexing notebooks.—All notebooks should be carefully indexed on the regulation blank 9–253 before they are forwarded to the Washington office. The index slip should be left loose in the book, in order

that the file clerk may more readily check the contents. It will then be pasted on the cover. The following shows a sample of the indexing:

9-253 DEPARTMENT OF THE INTERIOR.

UNITED STATES GEOLOGICAL SURVEY.

HYDROGRAPHIC RECORDS.

Kind of notes, *Current-Meter.*
File No., *5399.*
Date received, *February 1, 1904.*
Hydrographers, *Chandler, E. F., and Richards, R.*

Stream.	Locality.	State.	Date.	Page.
			1903.	
Heart	*Church's ranch (Richardton).*	*N. Dak*	*S. 5*	*8*
Little Missouri	*Medora*	"	"	*7*
Mouse	*Minot*	"	*S. 25, O. 5*	*9, 11–12*
Pembina	*Neche*	"	*S. 28*	*10*
Red Lake	*Crookston*	*Minn*	*O. 12*	*13–14*
" "	*E. Grand Forks*	"	*Ag. 4*	*6*
Red River of the North	*Grand Forks*	*N. Dak*	*Ag. 1, 4*	*1–3, 4*
" " " " "	" " *(above forks)*	"	*Ag. 1, 4*	*3, 5*

NOTE.—Rivers to be arranged in alphabetical order. The same entries occurring several times should be entered but once and the various pages given.

If references are to consecutive pages use dash, e. g., 9-12; if not to consecutive pages, use comma, e. g., 1, 3, 5, 8.

*Enter under the noun, except where the adjective is a recognized part of the name, thus, Platte, South fork; but North Platte, Little Missouri.

Kinds of reports.—Reports in general may be divided into two classes: First, the regular monthly reports of work accomplished; second, special reports transmitting data or information to the office. The regular monthly reports are intended to give the hydrographer an idea of the work that is being done and the progress that is being made. The reports containing data go into the office files as permanent records and are used in making up the publications. An effort should be made to maintain this division.

Use of maps and sketches.—In all reports, descriptions of stations, etc., maps and sketches should be freely used to show the location of river stations, the sections of the country visited, and other details. In case topographic sheets have been prepared for the region under investigation the hydrographer should supply himself with copies of these sheets and make use of such portions of them as cover the territory examined. Upon these maps special features should be shown. In the absence of topographic sheets land office and other maps should be used.

Report maps.—Each district hydrographer should prepare a map of his territory from the best maps available. This map should show,

with their names, the principal streams and other features on which work will be carried on. He should divide his territory into its principal drainage areas and indicate them on the map. When this division has been decided upon, it should be carefully adhered to, so that the measurements and other work can be referred to drainage areas. These maps may be prepared on tracing cloth, and from them the Washington office can make a paper negative, from which as many copies as desired can be supplied. This map is desired for use in reports. With each report a map should be transmitted showing the localities that have been investigated, which should be referred to by number, so that the chief engineer can at a glance see what is being done.

Monthly reports.—A monthly report of the district hydrographer is to be made up of—

(a) A brief written report stating what was done during the month and the condition of the work.

(b) Form 9–171, on which shall be given an alphabetical list of the river stations in each State in his district. Stations in different States should not be shown on the same sheet, as these sheets are filed by States, so that separate sheets should be prepared for separate States. In the column of remarks, such notes as "New bench mark established," "Station discontinued" (with date), "Station established" (with date), "Temporarily closed," etc., are to be given. Names of the stations which are temporarily closed should be kept on the list until they are discontinued.

(c) Form 9–245 is designed for use by each man under the district hydrographer. On this form should be briefly stated what the employee did on each day during the month. Such general statements as clerical work, field work, etc., are not sufficient. In preparing the monthly report it is suggested that the district hydrographer should have each man under him fill out form 9–245 and form 9–171 for the stations which he has visited. From these two forms the general monthly report can be taken.

Each field assistant and a distant hydrographer, when on field trips, should report to the district office immediately after visiting a station or after completing a reconnaissance or any piece of work, the general results of the work done, and such detailed information as is necessary in making up the monthly report for that district. This enables the district hydrographer to complete his report without waiting for the return of an assistant who is making an extended trip.

Resident hydrographer's monthly report.—The following is a sample form for monthly report of resident hydrographer:

ATLANTA, GA., *October 31, 1903.*

Mr. F. H. NEWELL, *Chief Engineer.*

DEAR SIR: The following is a report of hydrographic work carried on in the States of Alabama, Georgia, and Tennessee during the month of October, 1903:

The following men were engaged: Messrs. M. R. Hall, J. M. Giles, and O. P.

Hall of the regular force, and Messrs. W. G. Green and B. S. Drane of the temporary force. Their duties were as follows:

M. R. Hall had general supervision of the work, spent ten days in the office in general correspondence, preparation of descriptions, computation on meter notes, and miscellaneous duties. The remainder of the month was spent in making a reconnaissance of Flint River and its tributaries.

J. M. Giles spent the whole month in the field, making meter measurements, repairing gages, and establishing bench marks.

O. P. Hall spent the whole month in the office, plotting discharge measurements and making rating tables.

W. G. Green was employed for ten days during the month in making low-water measurements.

B. S. Drane was employed for twenty days during the month, ten of which were spent in the field assisting in the reconnaissance of Flint River; the remainder of the time was spent in the office working on the report of the work.

The accompanying lists show the distribution of the gaging stations in the various States; also the work done at each station. On the index map is indicated the location of the stations and of the localities investigated.

The accompanying reports and forms give a portion of the data mentioned in this report. The remaining data will follow in a few days.

Very respectfully,

M. R. HALL, *Hydrographer.*

9–171. DEPARTMENT OF THE INTERIOR.

UNITED STATES GEOLOGICAL SURVEY.

DIVISION OF HYDROGRAPHY.

Atlanta, Ga., October 31, 1903.

Mr. *F. H. Newell,*
In charge of Divison of Hydrography.

SIR: The following is a brief statement of the hydrographic work preformed during the month of *November*, 1903, under the direction of *M. R. Hall;*

RIVER STATIONS MAINTAINED, ESTABLISHED, OR DISCONTINUED.

Stream.	Station.	State.	No. of discharge measurements.	Remarks.
Alabama River	*Montgomery*	*Alabama*	0	*New B. M. established Nov. 15, 1903.*
Alabama River	*Selma*	"	1	
Black Warrior R.	*Cordova*	"	0	
Black Warrior R.	*Tuscaloosa*	"	0	*Discontinued Nov. 30, 1903.*
Cahaba River	*Centerville*	"	2	
Choccolocco Cr.	*Jenifer*	"	1	
Coosa River	*Riverside*	"	1	*Gage washed out, Nov. 10; replaced Nov. 20.*

Under "Remarks" state date of establishment or discontinuance, changes in gage, interruptions in observations, etc. A brief report covering any matters of interest occurring during the month, and form 9-245, should usually accompany this form. These make up the monthly reports of the resident hydrographers.

9–245. DEPARTMENT OF THE INTERIOR.

UNITED STATES GEOLOGICAL SURVEY.

HYDROGRAPHIC BRANCH.

Report of services rendered by *J. M. Giles* during month of *November*, 1905.

[To accompany vouchers form 9–009 or subvouchers 9–019, giving briefly the character of work performed each day.]

1. *Sunday.*
2. *Plotting discharge measurements, checking gage-height cards.*
3. *General correspondence and report on reconnaissance of Flint River.*
4. *Measured discharge of Alabama River at Salem, en route to Montgomery.*
5. *Measured discharge of Alabama River at Montgomery.*
6. *Computing discharge measurements and general correspondence.*
7. *Making up monthly reports.*

I certify that the services were rendered as above stated.

J. M. Giles.

Approved by *M. R. Hall.*

Reports on reconnaissance, surveys, investigations, etc.—As soon as practicable after the completion of the reconnaissance work, surveys, investigations, etc., a comprehensive report should be transmitted, stating concisely the information collected and the deductions which have been made, based upon the investigations. It is not desirable to accumulate data for a semiannual or yearly report. The material should be separated, as far as possible, into independent unit reports, and these should be transmitted as the work progresses.

Reporting new river stations.—When a river station is established a statement should be transmitted, together with a description of the station, on form 9–197, stating the general reasons for establishing the station and the conditions which led to its establishment at the particular site selected.

Authority for carrying on work.—At the beginning of each season or before starting work which involves a considerable expenditure of money, the hydrographer or engineer in charge should submit for the approval of the chief engineer a brief outline of the work which it is proposed to carry on. This should state the purpose for which the data are to be collected, the cost, and other details which will give the chief engineer an idea of what it is proposed to do. Approval of the plan is to be considered authority for carrying on such investigations as may be deemed necessary to accomplish the end desired.

Furnishing information to the public.—All data collected by Survey employees should be submitted to the Director through the hydrographer-in-charge before it is given to the public. In general, requests for data which are ready for distribution should be addressed to the

Director or the hydrographer-in-charge at Washington. When, however, requests are received by the district hydrographers or engineers they may furnish the information direct, if in so doing they are satisfied that the Survey will be in no way involved. Copies of all such requests and their answers should be sent to the hydrographer-in-charge for the Washington letter files. When the district hydrographer receives requests for data and he has some doubt concerning the advisability of complying with the request, he should prepare a letter containing the information desired, for the signature of the hydrographer-in-charge, who will decide whether the information should be given out. A carbon copy of the reply, with the answer and original request, should accompany the original request, so that a record of the data furnished may be kept in the Washington office.

Miscellaneous information.—It is expected that each district hydrographer and his assistants shall be on the lookout for any valuable information on the hydrography of the section in which they happen to be. This information should be systematically collected and carefully filed and indexed so as to be easily consulted. It is suggested that each employee supply himself with a small pocket case to hold 3 by 5 inch index cards. When in the field, items of importance may be noted on these cards, one subject to a card, and these, when filed in the office, will be a valuable source of information.

Publications containing progress reports of stream measurements.—The progress report of stream measurements for each year contains the results of the year's field work, including all original data, such as gage heights and discharge measurements, together with the office computations.

The following is a list of these publications by years:

1888. Tenth Annual Report, Part I.
1889. Eleventh Annual Report, Part II.
1890. Twelfth Annual Report, Part II.
1891. Thirteenth Annual Report, Part III.
1892. Fourteenth Annual Report, Part II.
1893. Bulletin No. 131.
1894. Sixteenth Annual Report, Part II. Bulletin No. 131.
1895. Seventeenth Annual Report, Part II. Bulletin No. 140.
1896. Eighteenth Annual Report, Part IV. Water-Supply Paper No. 11.
1897. Nineteenth Annual Report, Part IV. Water-Supply Papers Nos. 15, 16.
1898. Twentieth Annual Report, Part IV. Water-Supply Papers Nos. 27, 28.
1899. Twenty-first Annual Report, Part IV. Water-Supply Papers Nos. 35 to 39, inclusive.
1900. Twenty-second Annual Report, Part IV. Water-Supply Papers Nos. 47 to 52, inclusive.
1901. Water-Supply Papers Nos. 65, 66, and 75.
1902. Water-Supply Papers Nos. 82 to 85, inclusive.
1903. Water-Supply Papers Nos. 97 to 100, inclusive.

The supply of these publications is rapidly becoming exhausted, and many of them can be obtained only through members of Congress or by purchase from the Superintendent of Public Documents, Washington, D. C., or at some second-hand book store. District and resident hydrographers should endeavor to obtain a complete set of the above papers for their office files, and they and their assistants should familiarize themselves with their contents in order that they may know what work has been done by the Survey.

Miscellaneous hydrographic reports.—Aside from the publications containing yearly progress reports, mentioned above, special reports on various hydrographic subjects are prepared and published from time to time, in the series of Water-Supply and Irrigation Papers. These papers contain valuable information, and they should, as far as possible, be examined and studied by each man in the Survey, and special attention should be given to those relating to methods of the work.

COMPUTATIONS.

Rating curves and tables.—The object of taking gage heights and discharge measurements is to prepare rating curves from which a discharge table can be constructed. The practicability of applying a table depends on the general assumption that the discharge is a function of the gage height, and that for streams of practically constant cross section the discharge for the same gage height will be the same. By plotting the discharge measurements as abscissas and the corresponding gage heights as ordinates a discharge curve for the station can be drawn which will form a basis for the rating table. In preparing the rating curve a careful study of all available data should be made, and it is here that notes in regard to the various conditions at the time measurements were made are of great value. In choosing the scale for plotting the curve the maximum gage height and discharge should be considered. Its size should be such that the discharge can be read from the curve to the required degree of accuracy, and it should also be such that the inclination of the rating curve will be approximately 45°, for when the curve becomes nearly horizontal it is more difficult to accurately take the discharges from it. It is desirable to plot the measurements for different years on the same sheet, in order that a comparative study may be made of them. In plotting the discharge measurements different colors or symbols may be used to distinguish between measurements for different years.

The first draft of the rating table is taken from the curve. The table is afterwards smoothed out by adjusting first, and if necessary second, differences. The difference between the discharges for successive tenths should either be constant or increasing, never decreasing, and as far as possible it should follow some general law. Whether in the table the discharge should be carried out to the nearest second-foot or only to

the nearest five or ten second-feet, should be determined by the conditions at the station.

Figs. 7 and 8 are opposite extreme cases of rating curves. Fig. 7 shows one constructed from measurements carefully taken, and it also shows that it is necessary to have measurements made between gage heights 5.2 and 6.5 between which points the curve is not well determined. Fig. 8 shows that either the conditions at the rating station are poor or that the measurements are not being carefully made. When such conditions as are shown on this curve are encountered the district hydrographer should make a careful investigation to determine some solution of the problem.

Computation of daily discharge, monthly means, run-off, etc.—The object of the rating table is to enable the computation of daily discharges, monthly means, and run-off to be made. For such computations form 9–192 has been prepared (see p. 45). Daily gage heights

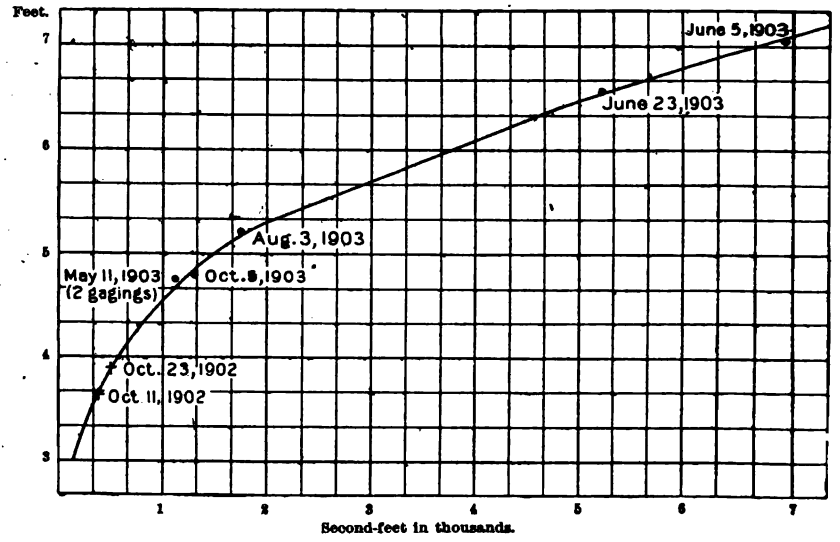

FIG. 7.—A good station-rating curve.

are copied from the gage-height cards as they are received from the hydrographer, and the corresponding discharge is taken from the rating table shown on page 44, which is made up from the curve shown in fig. 7. It is assumed that this discharge is the mean discharge in cubic feet for that day. The mean for each month is the mean of the average daily discharges in cubic feet per second. The run-off per square mile in second-feet (cubic feet per second) is obtained by dividing the mean monthly discharge by the number of square miles of drainage area; being a rate of flow it is independent of the number of days in the month. The run-off in inches is obtained from the run-off per square mile by the use of the table on pages 57–60. The run-off in acre-feet is obtained from the mean for the month by the use of

the table on pages 52–56. The maximum and minimum are the discharges for that day on which the mean discharge is the greatest or

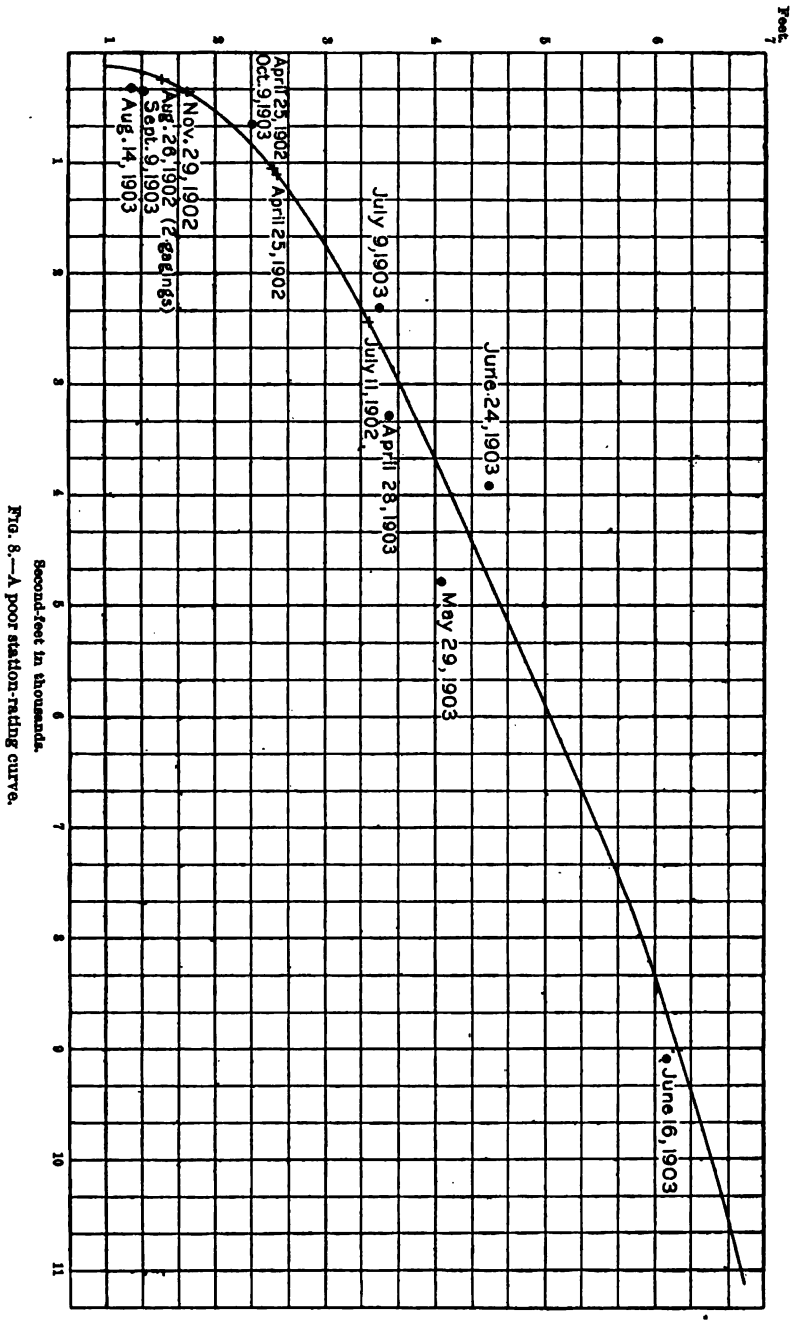

FIG. 8.—A poor station-rating curve.

least, respectively. When the daily discharge depends upon the mean of gage readings taken twice each day, the maximum or minimum

may not show the greatest or the least amount of water that was flowing in the river during the period covered by the records. They show the extreme discharges that may be considered as applicable to periods of twenty-four hours.

9–210.

DEPARTMENT OF THE INTERIOR.

Computed by F. H. B.
Checked by L. R. S.

UNITED STATES GEOLOGICAL SURVEY.

DIVISION OF HYDROGRAPHY.

RATING TABLE FOR STATION

For *Saint Mary* River at *International Line.* Constructed by *F. H. B.*, from discharge measurements number ——— to ———, as shown on accompanying blank form 9–207, and also from soundings made at intervening dates, as follows: ——— ———.

This table is applicable only from *September 3*, 1902, to *December 31*, 1903.

Gage height.	Discharge.	Difference.	Gage height.	Discharge.	Difference.	Gage height.	Discharge.	Difference.	Gage height.	Discharge.	Difference.
3.00	150		4.00	575	35	5.00	1,510	65	6.00	3,800	130
.05	162	12	.05	610	35	.05	1,575	65	.05	3,930	130
.10	175	13	.10	645	37	.10	1,640	80	.10	4,060	130
.15	190	15	.15	682	38	.15	1,720	80	.15	4,190	130
.20	205	15	.20	720	40	.20	1,800	100	.20	4,320	130
.25	222	17	.25	760	40	.25	1,900	100	.25	4,450	130
.30	239	17	.30	800	40	.30	2,000	120	.30	4,580	130
.35	257	18	.35	840	40	.35	2,120	120	.35	4,710	130
.40	275	20	.40	880	42	.40	2,240	130	.40	4,840	140
.45	295	20	.45	922	43	.45	2,370	130	.45	4,980	140
.50	315	22	.50	965	47	.50	2,500	130	.50	5,120	150
.55	337	23	.55	1,012	48	.55	2,630	130	.55	5,270	150
.60	360	25	.60	1,060	52	.60	2,760	130	.60	5,420	160
.65	385	25	.65	1,112	53	.65	2,890	130	.65	5,580	160
.70	410	25	.70	1,165	55	.70	3,020	130	.70	-------	-----
.75	435	25	.75	1,220	55	.75	3,150	130	.75	-------	-----
.80	460	27	.80	1,275	57	.80	3,280	130	.80	-------	-----
.85	487	28	.85	1,332	58	.85	3,410	130	.85	-------	-----
.90	515	30	.90	1,390	60	.90	3,540	130	.90	-------	-----
.95	545	30	.95	1,450	60	.95	3,670	130	.95	-------	-----

Remarks: *Lowest discharge measurement at gage height, 3.65 feet; highest measurement at gage height, 7.05 feet. Curve extended above and below these points. Tangent above gage height, 6.60 feet, with a difference of 160 per half tenth.*

9–192 DEPARTMENT OF THE INTERIOR. Computed by R. H. B. Checked by L. R. S.

UNITED STATES GEOLOGICAL SURVEY

DIVISION OF HYDROGRAPHY.

Daily mean gage height and discharge in second-feet of *St. Mary* River, *near International Line, Cardston, Alberta*, for 1903. Drainage area, *452* sq. miles.

Observer, *L. C. Shaw.*

Day.	January.		February.		March.		April.		May.		June.	
	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
1	4.65	1,112	4.40	880	4.70	1,165	5.70	3,020	4.40	880	6.45	4,980
2	4.60	1,060	4.30	800	4.60	1,060	5.55	2,830	4.40	880	6.80	6,060
3	4.50	965	4.25	760	4.68	1,144	5.25	1,900	4.45	922	7.00	6,700
4	4.60	1,060	4.35	840	4.70	1,165	5.10	1,640	4.60	1,060	7.00	6,700
5	4.65	1,112	4.35	840	4.60	1,060	5.00	1,510	4.60	1,060	7.05	6,860
6	4.60	1,060	4.50	965	4.50	965	5.00	1,510	4.70	1,165	7.05	6,860
7	4.30	800	4.40	880	4.55	1,012	4.90	1,390	4.80	1,275	7.05	6,860
8	4.50	965	4.40	880	4.58	1,042	4.90	1,275	4.85	1,332	7.15	7,180
9	4.40	880	4.35	840	4.60	1,060	4.85	1,332	4.80	1,275	7.00	6,700
10	4.35	840	4.35	840	4.50	965	4.50	965	4.80	1,275	7.00	6,700
11	4.20	720	4.45	922	4.48	948	4.25	760	4.80	1,275	6.90	6,380
12	4.30	800	4.40	880	4.35	840	3.80	480	4.80	1,275	6.80	6,060
13	4.30	800	3.70	410	4.60	1,060	3.60	360	4.95	1,450	6.90	6,380
14	4.45	922	4.40	880	4.80	1,275	3.63	375	4.90	1,390	6.90	6,380
15	4.50	965	4.3	800	4.80	1,275	3.90	515	5.00	1,510	6.85	6,220
16	4.50	965	4.65	1,112	5.00	1,510	4.50	965	5.10	1,640	6.80	6,060
17	4.25	760	4.55	1,012	4.90	1,390	4.55	1,012	5.30	2,000	7.00	6,700
18	4.05	610	4.60	1,060	4.70	1,165	4.60	1,060	5.30	2,000	6.95	6,540
19	4.10	645	4.65	1,112	4.65	1,112	4.50	965	5.40	2,240	6.90	6,380
20	4.00	575	4.50	965	4.70	1,165	4.40	880	5.40	2,240	6.90	6,380
21	4.10	645	4.55	1,012	4.70	1,165	4.30	800	5.40	2,240	6.80	6,060
22	4.20	720	4.65	1,112	4.60	1,060	4.35	840	5.30	2,000	6.70	5,740
23	4.00	575	4.60	1,060	4.68	1,144	4.35	840	5.40	2,240	6.55	5,270
24	4.25	760	4.70	1,165	4.60	1,060	4.30	800	5.45	2,370	6.55	5,270
25	4.30	800	4.63	1,091	4.60	1,060	4.45	922	5.45	2,370	6.40	4,840
26	4.20	720	4.00	575	4.70	1,165	4.50	965	5.70	3,020	6.40	4,840
27	4.30	800	4.45	922	4.80	1,275	4.40	880	5.95	3,670	6.40	4,840
28	4.55	1,012	4.70	1,165	4.83	1,309	4.40	880	5.75	3,150	6.65	5,580
29	4.60	1,060			5.70	3,020	4.40	880	5.60	2,760	6.85	6,220
30	4.60	1,060			5.80	3,280	4.35	840	5.70	3,020	6.70	5,740
31	4.50	965			5.75	3,150			5.75	3,150		
Total		26,733		25,780		41,066		33,171		58,134		183,480
Mean		862		921		1,325		1,106		1,875		6,116
Run-off in inches		1.907		2.038		2.931		2.447		4.148		13.531
Run-off per square mile		2.198		2.122		3.381		2.728		4.781		15.100
Acre-feet		53,002		51,150		81,471		65,812		115,290		363,927

Rules for rejecting redundant figures.—In reducing the number of significant figures, or the number of decimal places, by dropping the last figure, the following rules apply:

(a) When the figure in the place to be rejected is less than 5, drop it without changing the preceding figure. Example: 1,827.4 becomes 1,827.

(b) When the figure in the place to be rejected is greater that 5, drop it and increase the preceding figure by 1. Example: 1,827.6 becomes 1,828.

(c) When the figure in the place to be rejected is 5, and it is preceded by an even figure, drop the 5. Example: 1,828.5 becomes 1,828.

(d) When the figure in the place to be rejected is 5, and it is preceded by an odd figure, drop the 5 and increase the preceding odd figure by 1. Example: 1,827.5 becomes 1,828.

Units of measurement.—The volume flowing in a stream is usually expressed in cubic feet per second, briefly expressed as "second-feet." A second-foot may be defined as the body of water flowing in a stream 1 foot wide, 1 foot deep, at a rate of 1 foot per second.

There is in common use by miners and irrigators throughout the West another unit called the miners' inch. It is an indefinite quantity, but in most States is recognized as the fiftieth part of a second-foot.

The unit of capacity used in connection with storage is the acre-foot, and is equivalent to 43,560 cubic feet. It is a quantity that would cover an acre to a depth of 1 foot. There is a convenient relation between the second-foot and the acre-foot; one second-foot flowing for twenty-four hours will deliver 86,400 cubic feet, which equals 1.9835 acre-feet, or, approximately 2 acre-feet.

One million gallons is sometimes used as the unit of storage; there is a convenient relation between this unit and the acre-foot. One acre-foot equals 325,851.45 gallons, or somewhat less than one-third of a million gallons.

Computation of meter measurements.—Because of the fact that velocity is not constant in all parts of the cross section the quantity flowing through each square foot of the section is not the same, and it is necessary to divide the cross-section area into parts and compute the discharge through each part separately. The total discharge is then the sum of the discharge through all the parts. The size of these component parts depends on the rapidity of change of the velocity and the degree of accuracy required. For convenience these component parts are vertical strips, bounded on two sides by soundings.

Fig. 9 shows the cross section of the Saline River near Salina, Kans., on September 30, 1903, while the discharge measurement recorded on page 48 was being made. The soundings were taken at each 5 feet of width from the initial point and the velocity was observed at 0.6 depth below the surface in each of these verticals.

The discharge through each 5-foot strip might be computed separately, but the computations are shortened by finding the discharge through each double strip at a time. The mean depth and the mean velocity for the double strip of width 10 feet are found from the formula:

$$d'_m = \frac{a + 4b + c}{6} \quad (1); \quad V'_m = \frac{V_a + 4V_b + V_c}{6} \quad (2)$$

The discharge through the double strip is:

$$Q' = d'_m V'_m 2L = \left(\frac{a + 4b + c}{6} 2L\right)\left(\frac{V_a + 4V_b + V_c}{6}\right) \quad (3)$$

Formulas (1) and (2) are based on the assumption that the stream bed is a series of parabolic arcs, also that the horizontal velocity curves

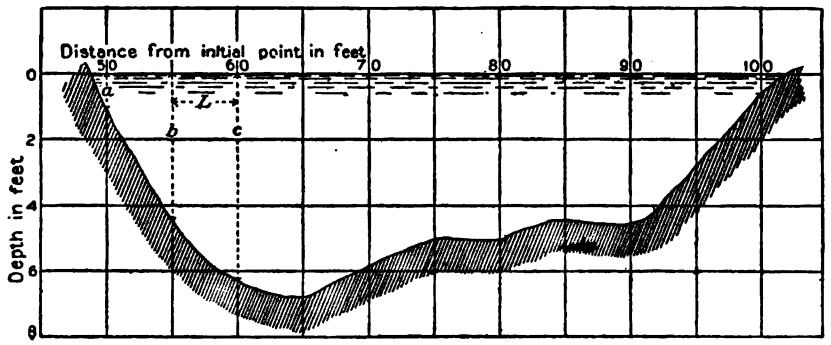

FIG. 9.—Cross section of Saline River at gaging station near Salina, Kans.

are parabolic arcs, both of which assumptions are approximately true at good current-meter stations.

d'_m = mean depth for double strip;

V'_m = mean velocity for double strip;

a, b, c are three consecutive depths, L feet apart;

V_a, V_b, V_c are observed velocities in the verticals, a, b, c;

L = the width of a single strip;

Q' = the discharge through double strip.

In computing the discharge and the mean depth through a single strip near the stream bank or a pier the mean velocity is found from the formulas:

$$V_m = \frac{V_c + V_a}{2}; \qquad d = \frac{a' + a}{2}$$

where either V_c or V_a and a' or a may be "0."

Velocity is computed to two places of decimals, mean depth, area, and discharge to one place of decimals for streams of ordinary size; for small streams with hard, smooth bottom, where the depth can be measured to hundredths foot, the mean depth and area should be computed to two places of decimals and the discharge to one place.

Gaging made *September 30*, 1903, by *E. C. Murphy*. Meter No. *388*, on *Saline* River near *Salina*, State of *Kansas*.

[Gage height: Beginning *7.96* ft., ending *7.96* ft., mean *7.96* feet. River *stationary*. Total area, *235* sq. feet. Mean velocity, *0.82*. Discharge, *190* second-feet.]

Dist. from initial point.	Observations.				Velocity computations.				Computations of—		Area of sec.	Discharge of section.	Remarks. (On condition of channel, wind, equipment, gage, boat, cable, methods, accuracy. Use cross-section pages in back of book for sketches.)
	Depth.	Depth of observat.	Time in seconds.	Revolutions.	Total number revolutions.	Revolutions per second.	Velocity per second.	Mean velocity per second.	Width.	Mean depth.			
49	*0.0*	.....	.....	.....	.....	.....	*0.0*	*0.20*	*1*	*0.6*	*0.6*	*0.1*	Length of gage wire measured and found to *be 35.47 feet.*
50	*1.1*	*0.7*	*50*	*7 and 8*	*15*	.....	*0.40*	.....	.....	.....	.....	.....	*Clear. No wind.*
55	*4.5*	*2.7*	*50*	*16 and 17*	*33*	.....	*0.82*	*0.79*	*10*	*4.2*	*42.0*	*33.2*	
60	*6.3*	*3.8*	*50*	*21 and 22*	*43*	.....	*1.06*	.....	.....	.....	.....	.....	
65	*6.8*	*4.1*	*50*	*19 and 18*	*37*	.....	*0.92*	*0.95*	*10*	*6.6*	*66.0*	*62.7*	
70	*5.9*	*3.5*	*50*	*19 and 20*	*39*	.....	*0.97*	.....	.....	.....	.....	.....	
75	*5.0*	*3.0*	*50*	*23 and 24*	*47*	.....	*1.15*	*1.11*	*10*	*5.2*	*52.0*	*57.7*	
80	*5.0*	*3.0*	*50*	*22 and 23*	*45*	.....	*1.10*	.....	.....	.....	.....	.....	
85	*4.4*	*2.6*	*50*	*14 and 15*	*29*	.....	*0.73*	*0.73*	*10*	*4.5*	*45.0*	*32.9*	
90	*4.5*	*2.7*	*50*	*6 and 6*	*12*	.....	*0.33*	.....	.....	.....	.....	.....	
95	*2.8*	*1.7*	*50*	*0 and 0*	*0*	.....	*0.00*	*0.17*	*5*	*3.6*	*18.0*	*3.1*	
100	*0.6*	*0.4*	*50*	*0 and 0*	*0*	.....	*0.00*	*0.00*	*7*	*1.3*	*9.1*	*0.0*	
102	*0.0*	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	*232.7*	*189.7*	

Computed by *E. C. Murphy*. Checked by *E. C. Murphy*.

If the district hydrographer desires he can compute discharge from the formula given on page eighty-six of "Instructions relating to the work of the United States Geological Survey, May 1, 1903," namely, (with altered symbols); $Q' = \left(\frac{a + 6b + c}{8}\right) LV_b$. The letters have the same meaning as in the formula on page 47, except that Q' is the discharge through the vertical section extending from halfway between a and b to halfway between b and c.

It sometimes happens that the velocity becomes very small or "0" in some parts of the cross-section at a station, as in the case of the Salina station for the part from 95 to 102 feet from the initial point. This area of small or "0" velocity is sometimes neglected in computing the cross-sectional area and the mean velocity. Mean velocity obtained in this way is sometimes misleading, because it may make a section in which the true velocity is only 0.3 or 0.4 foot per second (a section that should not be used) appear to have a velocity of half a foot or more per second. There is danger, too, that if this low velocity area is neglected in computing mean velocity, no attention will be given to measuring those velocities, and thus an error will be introduced into the discharge. It has been decided to include the whole area in the column headed "area" in computing mean velocity.

COMPUTATIONS OF VERTICAL-VELOCITY CURVES AND COEFFICIENTS.

The method of making vertical-velocity-curve observations is described on page 20, the form for recording the observations is given on page 50, the velocities in column 4 are plotted as shown on page 51, the depth of center of meter below the surface being used as ordinates and velocities as abscissas. A smooth curve is drawn among them making a graphic adjustment of the observations. The mean abscissa of this curve is the mean velocity in this vertical. The depth below the surface of the thread of mean velocity is the ordinate which corresponds to the mean abscissa or to the computed mean velocity.

To compute the mean velocity from the vertical-velocity curve, divide the depth into from 5 to 10 equal parts and write the velocity at the center of each part in column 6, headed "velocity from curve." Find the sum of these and divide by the number of parts; the quotient is the mean velocity in that vertical.

It is often more convenient, when the depth is a number of feet and a fraction, as 8.3 feet, to divide the depth into 8 parts of a foot width, and a part of 0.3 foot width. Then the velocity to enter in column 6 for the narrow part is 0.3 of the velocity at the center of it.

The velocities at a point 1 foot below the surface, mid-depth, and bottom are read directly from the vertical-velocity curve, and recorded in column 8. The "coefficients for reducing to mean velocity," required in column 9, are obtained by dividing the mean velocity by each of the velocities in column 8.

Vertical-velocity measurement made *November 2, 1903*, by E. C. Murphy, meter No. *585* feet from initial point

Gage height: beginning *3.08* ft., ending 3.08 ft.

FIELD NOTES.				DATA FROM CURVE AND COMPUTATION.				
Depth of center of meter below surface in feet.	Number revolutions per 50 seconds.	Number revolutions per second.	Velocity per second.	Middle of horizontal section.	Velocity from curve.	Point in vertical.	Velocity.	Coefficient for reducing to mean velocity.
0.5	*61–61* *61–62*	*1.22*	*2.85*	1	*2.85*	0.6 depth.	*2.45*	*1.01*
1.5	*59* *57*	*1.16*	*2.71*	2	*2.78*	1 ft. below surface.	*2.82*	*0.88*
2.5	*58* *61*	*1.19*	*2.79*	3	*2.72*	Bottom.	*1.40*	*1.77*
3.5	*54* *55*	*1.09*	*2.58*	4	*2.62*	Mid depth.	*2.57*	*0.96*
4.5	*57* *54*	*1.11*	*2.60*	5	*2.50*			
5.5	*46* *51*	*.96*	*2.25*	6	*2.35*	Depth of mean velocity=	*4.5* ft., *56* per cent of depth.	
6.5	*44* *48*	*.92*	*2.16*	7	*2.16*			
7.5	*38* *39*	*.77*	*1.82*	8	*1.82*			
8.0				9				
				10				
				Total..	*19.80*	Computed by *Brundage*.		
				Mean .	*2.48*	Checked by *Marsh*.		
				Fall of river, —— feet per mile.				
				Remarks.—(Wind conditions. Character of stream bed. Roughness under surface of ice, etc.)				

338, on *Susquehanna* River, at *Harrisburg*, State of *Pennsylvania*. Measurements at for soundings. Depth *8* ft.

mean *5.06* ft. Channel *open*. Thickness of ice, —— ft.

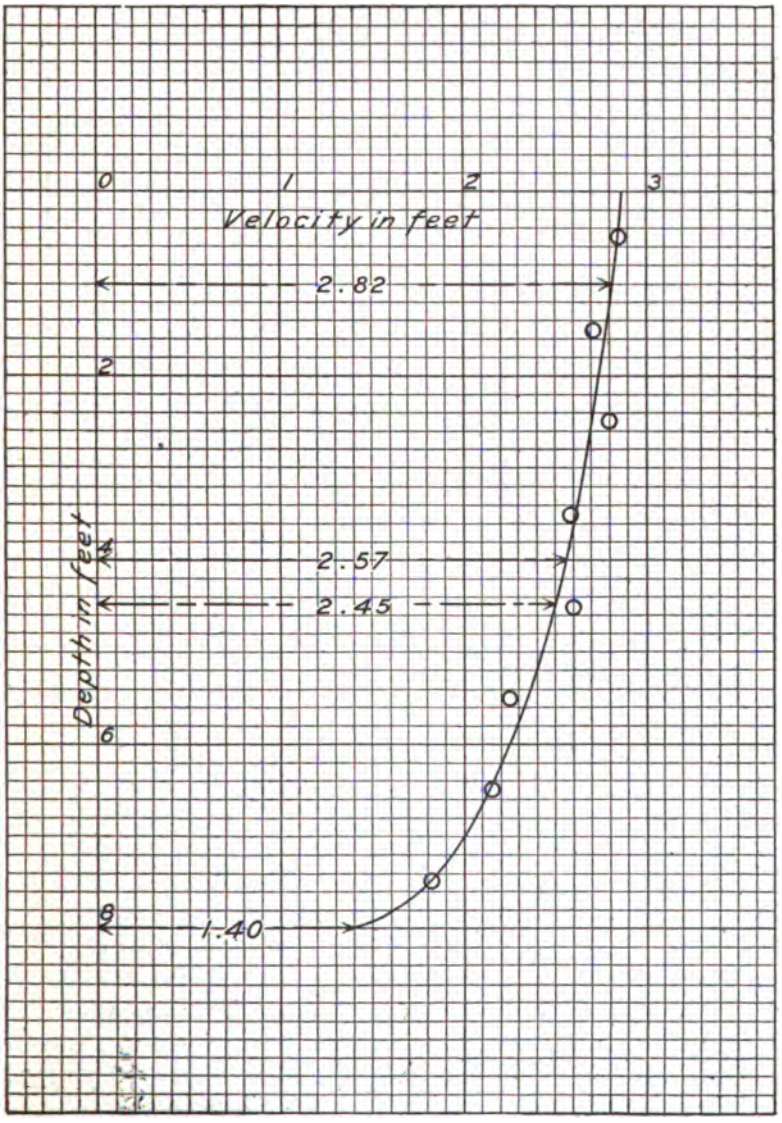

FIG. 10.—Vertical-velocity curve.

TABLES.

TABLES FOR COMPUTATION OF RUN-OFF.

Table for converting second-feet into acre-feet per day.

Days.	Second-feet.								
	1.	2.	3.	4.	5.	6.	7.	8.	9.
	Acre-ft.	*Acre-ft.*	*Acre-ft.*	*Acre-ft.*	*Acre-ft.*	*Acre-ft.*	*Acre-ft.*	*Acre-ft.*	*Acre-ft.*
1.....	1.98	3.97	5.95	7.93	9.92	11.90	13.88	15.87	17.85
2.....	3.97	7.93	11.90	15.87	19.83	23.80	27.77	31.74	35.70
3.....	5.95	11.90	17.85	23.80	29.75	35.70	41.65	47.60	53.55
4.....	7.93	15.87	23.80	31.74	39.67	47.60	55.54	63.47	71.40
5.....	9.92	19.83	29.75	39.67	49.59	59.50	69.42	79.34	89.26
6.....	11.90	23.80	35.70	47.60	59.50	71.40	83.31	95.21	107.11
7.....	13.88	27.77	41.65	55.54	69.42	83.31	97.19	111.07	124.96
8.....	15.87	31.74	47.60	63.47	79.34	95.21	111.07	126.94	142.81
9.....	17.85	35.70	53.55	71.40	89.26	107.11	124.96	142.81	160.66
10.....	19.83	39.67	59.50	79.34	99.17	119.01	138.84	158.68	178.51
11.....	21.82	43.64	65.45	87.27	109.09	130.91	152.73	174.55	196.41
12.....	23.80	47.60	71.40	95.21	119.01	142.81	166.61	190.41	214.21
13.....	25.79	51.57	77.35	103.14	128.93	154.71	180.50	206.28	232.07
14.....	27.77	55.54	83.31	111.07	138.84	166.61	194.38	222.15	249.92
15.....	29.75	59.50	89.26	119.01	148.76	178.51	208.26	238.02	267.77
16.....	31.74	63.47	95.21	126.94	158.68	190.41	222.15	253.88	285.62
17.....	33.72	67.44	101.16	134.88	168.59	202.31	236.03	269.75	303.47
18.....	35.70	71.40	107.11	142.81	178.51	214.21	249.92	285.62	321.32
19.....	37.69	75.37	113.06	150.74	188.43	226.12	263.80	301.49	339.17
20.....	39.67	79.34	119.01	158.68	198.35	238.02	277.69	317.36	357.02
21.....	41.65	83.31	124.96	166.61	208.26	249.92	291.57	333.22	374.87
22.....	43.64	87.27	130.91	174.55	218.18	261.82	305.45	349.09	392.73
23.....	45.62	91.24	136.86	182.48	228.10	273.72	319.34	364.96	410.58
24.....	47.60	95.21	142.81	190.41	238.02	285.62	333.22	380.82	428.43
25.....	49.59	99.17	148.76	198.35	247.93	297.52	347.11	396.69	446.28
26.....	51.57	103.14	154.71	206.28	257.85	309.42	360.99	412.56	464.13
27.....	53.55	107.11	160.66	214.21	267.77	321.32	374.88	428.43	481.98
28.....	55.54	111.07	166.61	222.15	277.69	333.22	388.76	444.30	499.83
29.....	57.52	115.04	172.56	230.08	287.60	345.12	402.64	460.17	517.69
30.....	59.50	119.01	178.51	238.02	297.52	357.02	416.53	476.03	535.54
31.....	61.49	122.98	184.46	245.95	307.44	368.93	430.41	491.90	553.39

As the months are of varying length it is necessary to use three or four factors to convert the average discharge for the month in second-feet into the total in acre-feet. One second-foot flowing for twenty-four hours is equivalent to 86,400 cubic feet. Since there are 43,560 square feet in an acre there will be the same number of cubic feet in an acre-foot. Dividing, it is found that 1 second-foot for twenty-four hours very nearly equals 2 acre-feet, or, in exact figures, 1.983471 acre-feet. This multiplied by the number of days in the month will give the total monthly discharge in acre-feet. This quantity, therefore, must be multiplied by 28 for the month of February, or 29 for that month in leap year, and by 30 or 31 for the other months.

For the month of February when it has 28 days the factor to be used is 55.537188. For convenience in computation this factor multiplied from 1 to 9 is given in the following table:

1	55.53719
2	111.07438
3	166.61156
4	222.14875
5	277.68594
6	333.22313
7	388.76032
8	444.29750
9	499.83469

When February has 29 days the factor to be used is 57.520659. This when multiplied from 1 to 9 gives the following:

1	57.52066
2	115.04132
3	172.56198
4	230.08264
5	287.60330
6	345.12395
7	402.64461
8	460.16527
9	517.68593

For the months containing 30 days, viz, April, June, September, and November, the factor to be used is 59.504130. This, when multiplied by the unit figures, gives the following results:

1	59.50413
2	119.00826
3	178.51239
4	238.01652
5	297.52065
6	357.02478
7	416.52891
8	476.03304
9	535.53717

For the months containing 31 days, viz, January, March, May, July, August, October, and December, the factor to be used is 61.487601. This, when multiplied by the unit figures, gives the following results:

1	61.48760
2	122.97520
3	184.46280
4	245.95040
5	307.43800
6	368.92561
7	430.41321
8	491.90081
9	553.38841

Table for converting second-feet into acre-feet per month of twenty-eight days.

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0.....	.....	55.537	111.074	166.612	222.149	277.686	333.223	388.760	444.298	499.835
1.....	555.372	610.909	666.446	721.983	777.521	833.058	888.595	944.132	999.669	1,055.206
2.....	1,110.744	1,166.281	1,221.818	1,277.355	1,332.892	1,388.430	1,443.967	1,499.504	1,555.041	1,610.578
3.....	1,666.116	1,721.653	1,777.190	1,832.727	1,888.264	1,943.802	1,999.339	2,054.876	2,110.413	2,165.950
4.....	2,221.488	2,277.025	2,332.562	2,388.099	2,443.636	2,499.173	2,554.711	2,610.248	2,665.785	2,721.322
5.....	2,776.859	2,832.396	2,887.934	2,943.471	2,999.008	3,054.545	3,110.082	3,165.620	3,221.157	3,276.694
6.....	3,332.231	3,387.768	3,443.306	3,498.843	3,554.380	3,609.917	3,665.454	3,720.992	3,776.529	3,832.066
7.....	3,887.603	3,943.140	3,998.678	4,054.215	4,109.752	4,165.289	4,220.826	4,276.363	4,331.901	4,387.438
8.....	4,442.975	4,498.512	4,554.049	4,609.587	4,665.124	4,720.661	4,776.198	4,831.735	4,887.272	4,942.810
9.....	4,998.347	5,053.884	5,109.421	5,164.958	5,220.496	5,276.033	5,331.570	5,387.107	5,442.644	5,498.182
10.....	5,553.719	5,609.256	5,664.793	5,720.330	5,775.868	5,831.405	5,886.942	5,942.479	5,998.016	6,053.553

Table for converting second-feet into acre-feet per month of twenty-nine days.

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0.....	.....	57.521	115.041	172.562	230.083	287.603	345.124	402.645	460.165	517.686
1.....	575.207	632.727	690.248	747.768	805.289	862.810	920.330	977.851	1,035.372	1,092.892
2.....	1,150.413	1,207.934	1,265.454	1,322.975	1,380.496	1,438.016	1,495.537	1,553.058	1,610.578	1,668.099
3.....	1,725.620	1,783.140	1,840.661	1,898.182	1,955.702	2,013.223	2,070.744	2,128.264	2,185.785	2,243.306
4.....	2,300.826	2,358.347	2,415.868	2,473.388	2,530.909	2,588.430	2,645.950	2,703.471	2,760.992	2,818.512
5.....	2,876.033	2,933.554	2,991.074	3,048.595	3,106.116	3,163.636	3,221.157	3,278.678	3,336.198	3,393.719
6.....	3,451.240	3,508.760	3,566.281	3,623.802	3,681.322	3,738.843	3,796.363	3,853.884	3,911.405	3,968.925

7	4,026.446	4,083.967	4,141.487	4,199.008	4,256.529	4,314.049	4,371.570	4,429.091	4,486.611	4,544.132
8	4,601.653	4,659.173	4,716.694	4,774.215	4,831.735	4,889.256	4,946.777	5,004.297	5,061.818	5,119.339
9	5,176.859	5,234.380	5,291.901	5,349.421	5,406.942	5,464.463	5,521.983	5,579.504	5,637.024	5,694.545
10	5,752.066	5,809.586	5,867.107	5,924.628	5,982.148	6,039.669	6,097.190	6,154.710	6,212.231	6,269.752

Table for converting second-feet into acre-feet per month of thirty days.

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	----------	59.504	119.008	178.512	238.016	297.521	357.025	416.529	476.033	535.537
1	595.041	654.545	714.050	773.554	833.058	892.562	952.066	1,011.570	1,071.074	1,130.578
2	1,190.083	1,249.587	1,309.091	1,368.595	1,428.099	1,487.603	1,547.107	1,606.612	1,666.116	1,725.620
3	1,785.124	1,844.628	1,904.132	1,963.636	2,023.140	2,082.644	2,142.149	2,201.653	2,261.157	2,320.661
4	2,380.165	2,439.669	2,499.173	2,558.678	2,618.182	2,677.686	2,737.190	2,796.694	2,856.198	2,915.702
5	2,975.206	3,034.711	3,094.215	3,153.719	3,213.223	3,272.727	3,332.231	3,391.735	3,451.240	3,510.744
6	3,570.248	3,629.752	3,689.256	3,748.760	3,808.264	3,867.768	3,927.272	3,986.777	4,046.281	4,105.785
7	4,165.289	4,224.793	4,284.297	4,343.801	4,403.306	4,462.810	4,522.314	4,581.818	4,641.322	4,700.826
8	4,760.330	4,819.834	4,879.339	4,938.843	4,998.347	5,057.851	5,117.355	5,176.859	5,236.363	5,295.868
9	5,355.372	5,414.876	5,474.380	5,533.884	5,593.388	5,652.892	5,712.396	5,771.901	5,831.405	5,890.909
10	5,950.413	6,009.917	6,069.421	6,128.925	6,188.430	6,247.934	6,307.438	6,366.942	6,426.446	6,485.950

Table for converting second-feet into acre-feet per month of thirty-one days.

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	----------	61.488	122.975	184.463	245.950	307.438	368.926	430.413	491.901	553.388
1	614.876	676.364	737.851	799.339	860.826	922.314	983.802	1,045.289	1,106.777	1,168.264
2	1,229.752	1,291.240	1,352.727	1,414.215	1,475.702	1,537.190	1,598.678	1,660.165	1,721.653	1,783.140
3	1,844.628	1,906.116	1,967.603	2,029.091	2,090.578	2,152.066	2,213.554	2,275.041	2,336.529	2,398.016
4	2,459.504	2,520.992	2,582.479	2,643.967	2,705.454	2,766.942	2,828.430	2,889.917	2,951.405	3,012.892
5	3,074.380	3,135.868	3,197.355	3,258.843	3,320.330	3,381.818	3,443.306	3,504.793	3,566.281	3,627.768
6	3,689.256	3,750.744	3,812.231	3,873.719	3,935.206	3,996.694	4,058.182	4,119.669	4,181.157	4,242.644
7	4,304.132	4,365.620	4,427.107	4,488.595	4,550.082	4,611.570	4,673.058	4,734.545	4,796.033	4,857.520
8	4,919.008	4,980.496	5,041.983	5,103.471	5,164.958	5,226.446	5,287.934	5,349.421	5,410.909	5,472.396
9	5,533.884	5,595.372	5,656.859	5,718.347	5,779.834	5,841.322	5,902.810	5,964.297	6,025.785	6,087.272
10	6,148.760	6,210.248	6,271.735	6,333.223	6,394.710	6,456.198	6,517.686	6,579.173	6,640.661	6,702.148

The depth of run-off over the drainage basin is usually computed in inches for convenience of comparison with the depth of rainfall, which is almost invariably given in that unit. This depth can most conveniently be computed from the run-off per square mile by computation based upon the number of days in each month and the relation between the rate of flow and the depth in inches for this quantity were it held during the given number of days. One second-foot for twenty-four hours is equivalent to 86,400 cubic feet in one day. In other words, 1 cubic foot per second run-off from 1 square mile would, if held upon this area, cover it to a depth represented by dividing 86,400 by the number of square feet in a mile, 27,878,400, or 5,280 squared. Completing this division, it is found that 1 second-foot for one day is equivalent to a body of water covering 1 square mile 0.003099174 feet, or 0.037190088 inch. Multiplying this by the number of days in a month gives the following factors:

28 days	1.041322528
29 days	1.078512604
30 days	1.115702680
31 days	1.152892756

Table for converting run-off in second-feet per square mile into depth in inches per month of twenty-eight days.

	0.0.	0.1.	0.2.	0.3.	0.4.	0.5.	0.6.	0.7.	0.8.	0.9.
1.....	1.0413	1.1454	1.2496	1.3537	1.4578	1.5620	1.6661	1.7702	1.8744	1.9785
2.....	2.0826	2.1868	2.2909	2.3950	2.4992	2.6033	2.7074	2.8116	2.9157	3.0198
3.....	3.1240	3.2281	3.3322	3.4364	3.5405	3.6446	3.7488	3.8529	3.9570	4.0612
4.....	4.1653	4.2694	4.3736	4.4777	4.5818	4.6859	4.7901	4.8942	4.9983	5.1025
5.....	5.2066	5.3107	5.4149	5.5190	5.6231	5.7273	5.8314	5.9355	6.0397	6.1438
6.....	6.2479	6.3521	6.4562	6.5603	6.6645	6.7686	6.8727	6.9769	7.0810	7.1851
7.....	7.2892	7.3934	7.4975	7.6016	7.7058	7.8100	7.9140	8.0182	8.1223	8.2264
8.....	8.3306	8.4347	8.5388	8.6430	8.7471	8.8512	8.9554	9.0595	9.1636	9.2678
9.....	9.3719	9.4760	9.5802	9.6843	9.7884	9.8926	9.9967	10.1008	10.2050	10.3091
10.....	10.4132	10.5174	10.6215	10.7256	10.8297	10.9339	11.0380	11.1421	11.2463	11.3504

Table for converting run-off in second-feet per square mile into depth in inches per month of twenty-nine days.

	0.0.	0.1.	0.2.	0.3.	0.4.	0.5.	0.6.	0.7.	0.8.	0.9.
1.....	1.0785	1.1864	1.2942	1.4021	1.5099	1.6178	1.7256	1.8335	1.9413	2.0492
2.....	2.1570	2.2649	2.3727	2.4806	2.5884	2.6963	2.8041	2.9120	3.0198	3.1277
3.....	3.2355	3.3434	3.4512	3.5591	3.6669	3.7748	3.8826	3.9905	4.0983	4.2062
4.....	4.3140	4.4219	4.5298	4.6376	4.7454	4.8533	4.9612	5.0690	5.1768	5.2847
5.....	5.3926	5.5004	5.6083	5.7161	5.8240	5.9318	6.0397	6.1475	6.2554	6.3632
6.....	6.4711	6.5789	6.6868	6.7946	6.9025	7.0103	7.1182	7.2260	7.3339	7.4417
7.....	7.5496	7.6574	7.7653	7.8731	7.9810	8.0888	8.1967	8.3045	8.4124	8.5202

8.....	8.6281	8.7359	8.8438	8.9516	9.0595	9.1674	9.2752	9.3830	9.4909	9.5988
9.....	9.7066	9.8145	9.9223	10.0302	10.1380	10.2459	10.3537	10.4616	10.5694	10.6773
10.....	10.7851	10.8930	11.0008	11.1087	11.2165	11.3244	11.4322	11.5401	11.6479	11.7558

Table for converting run-off in second-feet per square mile into depth in inches per month of thirty days.

	0.0.	0.1.	0.2.	0.3.	0.4.	0.5.	0.6.	0.7.	0.8.	0.9.
1.....	1.1157	1.2273	1.3388	1.4504	1.5620	1.6736	1.7851	1.8967	2.0083	2.1198
2.....	2.2314	2.3430	2.4545	2.5661	2.6777	2.7893	2.9008	3.0124	3.1240	3.2355
3.....	3.3471	3.4587	3.5702	3.6818	3.7934	3.9050	4.0165	4.1281	4.2397	4.3512
4.....	4.4628	4.5744	4.6859	4.7975	4.9091	5.0207	5.1322	5.2438	5.3554	5.4669
5.....	5.5785	5.6901	5.8016	5.9132	6.0248	6.1364	6.2479	6.3595	6.4711	6.5826
6.....	6.6942	6.8058	6.9174	7.0289	7.1405	7.2521	7.3636	7.4752	7.5868	7.6983
7.....	7.8099	7.9215	8.0330	8.1446	8.2562	8.3678	8.4793	8.5909	8.7025	8.8140
8.....	8.9256	9.0372	9.1488	9.2603	9.3719	9.4835	9.5950	9.7066	9.8182	9.9298
9.....	10.0413	10.1529	10.2645	10.3760	10.4876	10.5992	10.7107	10.8223	10.9339	11.0454
10.....	11.1570	11.2686	11.3802	11.4917	11.6033	11.7149	11.8264	11.9380	12.0496	12.1612

Table for converting run-off in second-feet per square mile into depth in inches per month of thirty-one days.

	0.0.	0.1.	0.2.	0.3.	0.4.	0.5.	0.6.	0.7.	0.8.	0.9.
1.....	1.1529	1.2682	1.3835	1.4988	1.6140	1.7293	1.8446	1.9599	2.0752	2.1905
2.....	2.3058	2.4211	2.5364	2.6516	2.7669	2.8822	2.9975	3.1128	3.2281	3.3434
3.....	3.4587	3.5740	3.6892	3.8045	3.9198	4.0351	4.1504	4.2657	4.3810	4.4963
4.....	4.6116	4.7268	4.8421	4.9574	5.0727	5.1880	5.3033	5.4186	5.5339	5.6492
5.....	5.7645	5.8798	5.9950	6.1103	6.2256	6.3409	6.4562	6.5715	6.6868	6.8021
6.....	6.9174	7.0326	7.1479	7.2632	7.3785	7.4938	7.6091	7.7244	7.8397	7.9550
7.....	8.0702	8.1855	8.3008	8.4161	8.5314	8.6467	8.7620	8.8773	8.9926	9.1078
8.....	9.2231	9.3384	9.4537	9.5690	9.6843	9.7996	9.9149	10.0302	10.1454	10.2607
9.....	10.3760	10.4913	10.6066	10.7219	10.8372	10.9525	11.0678	11.1830	11.2983	11.4136
10.....	11.5289	11.6442	11.7595	11.8748	11.9901	12.1054	12.2206	12.3359	12.4512	12.5665

MISCELLANEOUS TABLES.

Cubic feet into gallons.

1 cubic foot = 1,728 cubic inches = $\frac{1,728}{231}$ gallons = 7.4805194 gallons.

[In gallons.]

1 cubic foot	7.4805194	6 cubic feet	44.8831164
2 cubic feet	14.9610388	7 cubic feet	52.3636358
3 cubic feet	22.4415582	8 cubic feet	59.8441552
4 cubic feet	29.9220776	9 cubic feet	67.3246746
5 cubic feet	37.4025970		

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	.....	7.48	14.96	22.44	29.92	37.40	44.88	52.36	59.84	67.32
1	74.81	82.28	89.77	97.25	104.73	112.21	119.69	127.17	134.65	142.13
2	149.61	157.09	164.57	172.05	179.53	187.01	194.49	201.97	209.45	216.94
3	224.42	231.90	239.38	246.86	254.34	261.82	269.30	276.78	284.26	291.74
4	299.22	306.70	314.18	321.66	329.14	336.62	344.10	351.58	359.06	366.55
5	374.03	381.51	388.99	396.47	403.95	411.43	418.91	426.39	433.87	441.35
6	448.83	456.31	463.79	471.27	478.75	486.23	493.71	501.19	508.68	516.16
7	523.64	531.12	538.60	546.08	553.56	561.04	568.52	576.00	583.48	590.96
8	598.44	605.92	613.40	620.88	628.36	635.84	643.32	650.81	658.29	665.77
9	673.25	680.73	688.21	695.69	703.17	710.65	718.13	725.61	733.09	740.57

Gallons into cubic feet.

1 United States liquid gallon = 231 cubic inches = $\frac{231}{1,728}$ cubic foot = 0.133680555 cubic feet.

[In cubic feet.]

1 gallon	0.13368055	6 gallons	0.80208330
2 gallons	.26736110	7 gallons	.93576385
3 gallons	.40104165	8 gallons	1.06944440
4 gallons	.53472220	9 gallons	1.20312495
5 gallons	.66840275		

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	.....	0.1337	0.2674	0.4010	0.5347	0.6684	0.8021	0.9358	1.0694	1.2031
1	1.3368	1.4705	1.6042	1.7378	1.8715	2.0052	2.1389	2.2726	2.4062	2.5399
2	2.6736	2.8073	2.9410	3.0746	3.2083	3.3420	3.4757	3.6094	3.7430	3.8767
3	4.0104	4.1441	4.2778	4.4114	4.5451	4.6788	4.8125	4.9462	5.0799	5.2135
4	5.3472	5.4809	5.6146	5.7483	5.8819	6.0156	6.1493	6.2830	6.4167	6.5503
5	6.6840	6.8177	6.9514	7.0851	7.2187	7.3524	7.4861	7.6198	7.7535	7.8872
6	8.0208	8.1545	8.2882	8.4219	8.5556	8.6892	8.8229	8.9566	9.0901	9.2240
7	9.3576	9.4913	9.6250	9.7587	9.8924	10.0260	10.1597	10.2934	10.4271	10.5608
8	10.6944	10.8281	10.9618	11.0955	11.2292	11.3628	11.4965	11.6302	11.7639	11.8976
9	12.0312	12.1649	12.2986	12.4323	12.5660	12.6996	12.8333	12.9670	13.1007	13.2344

Feet per second into miles per hour.

1 foot per second = 3,600 feet per hour = $\frac{3,600}{5,280}$ or $\frac{15}{22}$ miles per hour = 0.681818.

[In miles per hour.]

1 foot per second	0.68182	6 feet per second	4.09091
2 feet per second	1.36364	7 feet per second	4.77273
3 feet per second	2.04545	8 feet per second	5.45455
4 feet per second	2.72727	7 feet per second	6.13636
5 feet per second	3.40909		

(See Smithsonian Meteorological Tables, No. 52.)

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	.....	0.6818	1.3636	2.0455	2.7273	3.4091	4.0909	4.7727	5.4546	6.1364
1	6.8182	7.5000	8.1818	8.8637	9.5455	10.2273	10.9091	11.5909	12.2728	12.9546
2	13.6364	14.3182	15.0000	15.6819	16.3637	17.0455	17.7273	18.4091	19.0910	19.7728
3	20.4546	21.1364	21.8182	22.5001	23.1819	23.8637	24.5455	25.2273	25.9092	26.5910
4	27.2728	27.9546	28.6364	29.3183	30.0001	30.6819	31.3637	32.0455	32.7274	33.4092
5	34.0910	34.7728	35.4546	36.1365	36.8183	37.5001	38.1819	38.8637	39.5456	40.2274
6	40.9092	41.5910	42.2728	42.9547	43.6365	44.3183	45.0001	45.6819	46.3638	47.0456
7	47.7274	48.4092	49.0910	49.7729	50.4547	51.1365	51.8183	52.5001	53.1820	53.8638
8	54.5456	55.2274	55.9092	56.5911	57.2729	57.9547	58.6365	59.3183	60.0002	60.6820
9	61.3638	62.0456	62.7274	63.4093	64.0911	64.7729	65.4547	66.1365	66.8184	67.5002

Miles per hour into feet per second.

1 mile per hour = 5,280 feet per hour = $\frac{5,280}{3,600}$ or $\frac{22}{15}$ feet per second = 1.46667.

[In feet per second.]

1 mile per hour	1.46667	6 miles per hour	8.80000
2 miles per hour	2.93333	7 miles per hour	10.26667
3 miles per hour	4.40000	8 miles per hour	11.73333
4 miles per hour	5.86667	9 miles per hour	13.20000
5 miles per hour	7.33333		

(See Smithsonian Meteorological Tables, No 51.)

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	.....	1.4667	2.9333	4.4000	5.8667	7.3333	8.8000	10.2667	11.7333	13.2000
1	14.6667	16.1333	17.6000	19.0667	20.5333	22.0000	23.4667	24.9333	26.4000	27.8667
2	29.3333	30.8000	32.2667	33.7333	35.2000	36.6667	38.1333	39.6000	41.0667	42.5333
3	44.0000	45.4667	46.9333	48.4000	49.8667	51.3333	52.8000	54.2667	55.7333	57.2000
4	58.6667	60.1333	61.6000	63.0667	64.5333	66.0000	67.4667	68.9333	70.4000	71.8667
5	73.3333	74.8000	76.2667	77.7333	79.2000	80.6667	82.1333	83.6000	85.0667	86.5333
6	88.0000	89.4667	90.9333	92.4000	93.8667	95.3333	96.8000	98.2667	99.7333	101.2000
7	102.6667	104.1333	105.6000	107.0667	108.5333	110.0000	111.4667	112.9333	114.4000	115.8667
8	117.3333	118.8000	120.2667	121.7333	123.2000	124.6667	126.1333	127.6000	129.0667	130.5333
9	132.0000	133.4667	134.9333	136.4000	137.8667	139.3333	140.8000	142.2667	143.7333	145.2000

Second-feet per day into millions of gallons.

1 second-foot, or 7.4805194 gallons per second for 1 day, or 86,400 seconds=646,316.87616 gallons.

[In gallons.]

1 second-foot for 24 hours	646,316.87	6 second-feet for 24 hours	3,877,901.26
2 second-feet for 24 hours	1,292,633.75	7 second-feet for 24 hours	4,524,218.13
3 second-feet for 24 hours	1,938,950.63	8 second-feet for 24 hours	5,170,535.01
4 second-feet for 24 hours	2,585,267.50	9 second-feet for 24 hours	5,816,851.88
5 second-feet for 24 hours	3,231,584.38		

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	.....	0.6463	1.2926	1.9390	2.5853	3.2316	3.8779	4.5242	5.1705	5.8169
1	6.4632	7.1095	7.7558	8.4021	9.0484	9.6948	10.3411	10.9874	11.6337	12.2800
2	12.9263	13.5726	14.2190	14.8653	15.5116	16.1579	16.8042	17.4506	18.0969	18.7432
3	19.3895	20.0358	20.6821	21.3285	21.9748	22.6211	23.2674	23.9137	24.5600	25.2064
4	25.8527	26.4990	27.1453	27.7916	28.4379	29.0843	29.7306	30.3769	31.0232	31.6695
5	32.3158	32.9622	33.6085	34.2548	34.9011	35.5474	36.1937	36.8401	37.4864	38.1327
6	38.7790	39.4253	40.0716	40.7180	41.3643	42.0106	42.6569	43.3032	43.9495	44.5959
7	45.2422	45.8885	46.5348	47.1811	47.8274	48.4738	49.1201	49.7664	50.4127	51.0590
8	51.7054	52.3517	52.9980	53.6443	54.2906	54.9369	55.5833	56.2296	56.8759	57.5222
9	58.1685	58.8148	59.4612	60.1075	60.7538	61.4001	62.0464	62.6927	63.3391	63.9854

Millions of gallons into second-feet per day.

1 million gallons per 24 hours=$\frac{231,000,000}{1728\times86400}$ cubic feet per second, or 1.5472286 second-feet.

[In second-feet per 24 hours.]

1 million gallons	1.5472286	6 million gallons	9.2833716
2 million gallons	3.0944572	7 million gallons	10.8306002
3 million gallons	4.6416858	8 million gallons	12.3778288
4 million gallons	6.1889144	9 million gallons	13.9250574
5 million gallons	7.7361430		

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	.....	1.5472	3.0945	4.6417	6.1889	7.7361	9.2834	10.8306	12.3778	13.9251
1	15.4723	17.0195	18.5667	20.1140	21.6612	23.2084	24.7556	26.3029	27.8501	29.3973
2	30.9446	32.4918	34.0390	35.5862	37.1335	38.6807	40.2279	41.7752	43.3224	44.8696
3	46.4169	47.9641	49.5113	51.0585	52.6058	54.1530	55.7002	57.2474	58.7947	60.3419
4	61.8891	63.4364	64.9836	66.5308	68.0781	69.6253	71.1725	72.7197	74.2670	75.8142
5	77.3614	78.9087	80.4559	82.0031	83.5503	85.0976	86.6448	88.1920	89.7393	91.2865
6	92.8337	94.3809	95.9282	97.4754	99.0226	100.5699	102.1171	103.6643	105.2115	106.7588
7	108.3060	109.8532	111.4005	112.9477	114.4949	116.0421	117.5894	119.1366	120.6838	122.2311
8	123.7783	125.3255	126.8727	128.4200	129.9672	131.5144	133.0617	134.6089	136.1561	137.7033
9	139.2506	140.7978	142.3450	143.8923	145.4395	146.9867	148.5339	150.0812	151.6284	153.1756

Second-feet per day into acre-feet.

1 second-foot flow for one day = 86,400 cubic feet = $\frac{86,400}{43,560}$, or 1.983471 acre-feet.

[In acre-feet.]

1 second-foot for 24 hours	1.98347	6 second-feet for 24 hours	11.90083
2 second-feet for 24 hours	3.96694	7 second-feet for 24 hours	13.88430
3 second-feet for 24 hours	5.95041	8 second-feet for 24 hours	15.86777
4 second-feet for 24 hours	7.93388	9 second-feet for 24 hours	17.85124
5 second-feet for 24 hours	9.91735		

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	.....	1.983	3.967	5.950	7.934	9.917	11.901	13.884	15.868	17.851
1	19.835	21.818	23.802	25.785	27.769	29.752	31.736	33.719	35.702	37.686
2	39.669	41.653	43.636	45.620	47.603	49.587	51.570	53.554	55.537	57.521
3	59.504	61.488	63.471	65.455	67.438	69.421	71.405	73.388	75.372	77.355
4	79.339	81.322	83.306	85.289	87.273	89.256	91.240	93.223	95.207	97.190
5	99.174	101.157	103.140	105.124	107.107	109.091	111.074	113.058	115.041	117.025
6	119.008	120.992	122.975	124.959	126.942	128.926	130.909	132.892	134.876	136.859
7	138.843	140.826	142.810	144.793	146.777	148.760	150.744	152.728	154.711	156.694
8	158.678	160.661	162.645	164.628	166.611	168.595	170.578	172.562	174.545	176.529
9	178.512	180.496	182.479	184.463	186.446	188.430	190.413	192.397	194.380	196.364

Acre-feet into second-feet flow for 24 hours.

1 acre-foot each 24 hours = 43,560 cubic feet each 86,400 seconds = $\frac{43,560}{86,400}$, or $\frac{121}{240}$ second-foot flow for 24 hours = 0.50416666 +.

[In second-feet for 24 hours.]

1 acre-foot	0.50417	6 acre-feet	3.02500
2 acre-feet	1.00833	7 acre-feet	3.52917
3 acre-feet	1.51250	8 acre-feet	4.03334
4 acre-feet	2.01667	9 acre-feet	4.53750
5 acre-feet	2.52084		

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	.....	0.504	1.008	1.513	2.017	2.521	3.025	3.529	4.033	4.538
1	5.042	5.546	6.050	6.554	7.058	7.563	8.067	8.571	9.075	9.579
2	10.083	10.588	11.092	11.596	12.100	12.604	13.108	13.613	14.117	14.621
3	15.125	15.629	16.133	16.638	17.142	17.646	18.150	18.654	19.158	19.663
4	20.167	20.671	21.175	21.679	22.183	22.688	23.192	23.696	24.200	24.704
5	25.209	25.713	26.217	26.721	27.225	27.729	28.234	28.738	29.242	29.746
6	30.250	30.754	31.259	31.763	32.267	32.771	33.275	33.779	34.284	34.788
7	35.292	35.796	36.300	36.804	37.309	37.813	38.317	38.821	39.325	39.829
8	40.334	40.838	41.342	41.846	42.350	42.854	43.359	43.863	44.367	44.871
9	45.375	45.880	46.384	46.888	47.392	47.896	48.400	48.905	49.409	49.913

Acre-feet into millions of gallons.

1 acre-foot=43,560 cubic feet=$\frac{43,560\times1,728}{231}$, or $\frac{75,271,680}{231}$, or 325,851.428 gallons.

[In gallons.]

	Millions.	Thousands.		Millions.	Thousands.
1 acre-foot	0.325851428	325.9	6 acre-feet	1.95510858	1,955.1
2 acre-feet	.65170286	651.7	7 acre-feet	2.28096001	2,281.0
3 acre-feet	.97755429	977.6	8 acre-feet	2.60681144	2,606.8
4 acre-feet	1.30340572	1,303.4	9 acre-feet	2.93266287	2,932.7
5 acre-feet	1.62925715	1,629.3			

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	.....	0.3259	0.6517	0.9776	1.3034	1.6293	1.9551	2.2810	2.6068	2.9327
1	3.2585	3.5844	3.9102	4.2361	4.5619	4.8878	5.2136	5.5395	5.8653	6.1912
2	6.5170	5.8429	7.1687	7.4946	7.8204	8.1463	8.4721	8.7980	9.1238	9.4497
3	9.7755	10.1014	10.4272	10.7531	11.0789	11.4048	11.7306	12.0565	12.3823	12.7082
4	13.0341	13.3599	13.6857	14.0116	14.3374	14.6633	14.9891	15.3150	15.6408	15.9667
5	16.2926	16.6184	16.9443	17.2701	17.5960	17.9218	18.2477	18.5735	18.8994	19.2252
6	19.5511	19.8769	20.2028	20.5286	20.8545	21.1803	21.5062	21.8320	22.1579	22.4837
7	22.8096	23.1354	23.4613	23.7871	24.1130	24.4388	24.7647	25.0905	25.4164	25.7422
8	26.0681	26.3939	26.7198	27.0456	27.3715	27.6973	28.0232	28.3490	28.6749	29.0007
9	29.3266	29.6524	29.9783	30.3041	30.6300	30.9558	31.2817	31.6075	31.9334	32.2592

Millions of gallons into acre-feet.

One million United States liquid gallons or 231 million cubic inches = 133,680,555 cubic feet, or $\frac{133,680}{43,560}$ = 3.0688832 acre-feet.

[In acre-feet.]

1 million gallons	3.0688832	6 million gallons	18.4132992
2 million gallons	6.1377664	7 million gallons	21.4821824
3 million gallons	9.2066496	8 million gallons	24.5510656
4 million gallons	12.2755328	9 million gallons	27.6199488
5 million gallons	15.3444160		

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	.....	3.069	6.138	9.207	12.276	15.344	18.413	21.482	24.551	27.620
1	30.689	33.758	36.827	39.895	42.964	46.033	49.102	52.171	55.240	58.309
2	61.378	64.446	67.515	70.584	73.653	76.722	79.791	82.860	85.929	88.998
3	92.066	95.135	98.204	101.273	104.342	107.411	110.480	113.549	116.618	119.686
4	122.755	125.824	128.893	131.962	135.031	138.100	141.169	144.238	147.306	150.375
5	153.444	156.513	159.582	162.651	165.720	168.789	171.857	174.926	177.995	181.064
6	184.133	187.202	190.271	193.340	196.409	199.477	202.546	205.615	208.684	211.753
7	214.822	217.891	220.960	224.028	227.097	230.166	233.235	236.304	239.373	242.442
8	245.511	248.580	251.648	254.717	257.786	260.855	263.924	266.993	270.062	273.131
9	276.199	279.268	282.337	285.406	288.475	291.544	294.613	297.682	300.751	303.819

Second-feet into minute-gallons.

Factors: 1 cubic foot contains 1,728 cubic inches; 1 gallon has a capacity of 231 cubic inches; 1 second-foot equals [(1,728 ÷ 231) × 60] gallons per minute, or 448.831164 minute-gallons.

[In gallons per minute.]

1 second-foot	448.831164	6 second-feet	2,692.986984
2 second-feet	897.662328	7 second-feet	3,141.818148
3 second-feet	1,346.493492	8 second-feet	3,590.649312
4 second-feet	1,795.324656	9 second-feet	4,039.480476
5 second-feet	2,244.155820		

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	.....	449	898	1,346	1,795	2,244	2,693	3,142	3,591	4,039
1	4,488	4,937	5,386	5,835	6,284	6,732	7,181	7,630	8,079	8,528
2	8,977	9,425	9,874	10,323	10,772	11,221	11,670	12,118	12,567	13,016
3	13,465	13,914	14,363	14,811	15,260	15,709	16,158	16,607	17,056	17,504
4	17,953	18,402	18,851	19,300	19,749	20,197	20,646	21,095	21,544	21,993
5	22,442	22,890	23,339	23,788	24,237	24,686	25,135	25,583	26,032	26,481
6	26,930	27,379	27,828	28,276	28,725	29,174	29,623	30,072	30,521	30,969
7	31,418	31,867	32,316	32,765	33,214	33,662	34,111	34,560	35,009	35,458
8	35,906	36,355	36,804	37,253	37,702	38,151	38,599	39,048	39,497	39,946
9	40,395	40,844	41,292	41,741	42,190	42,639	43,088	43,537	43,985	44,434

Minute-gallons into second-feet.

Factors: 1 gallon contains 231 cubic inches; 1 cubic foot contains 1,728 cubic inches; 1 gallon per minute equals [(231 ÷ 1,728) ÷ 60] second-feet, or .002,228,009, 2 second-feet.

[In second-feet.]

1 minute-gallon	0.002,228,009	6 minute-gallons	0.013,368,055
2 minute-gallons	.004,456,018	7 minute-gallons	.015,596,064
3 minute-gallons	.006,684,028	8 minute-gallons	.017,824,074
4 minute-gallons	.008,912,037	9 minute-gallons	.020,052,083
5 minute-gallons	.011,140,046		

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	.....	0.0022	0.0045	0.0067	0.0089	0.0111	0.0134	0.0156	0.0178	0.0201
1	0.0223	.0245	.0267	.0290	.0312	.0334	.0356	.0379	.0401	.0423
2	.0446	.0468	.0490	.0512	.0535	.0557	.0579	.0602	.0624	.0646
3	.0668	.0691	.0713	.0735	.0758	.0780	.0802	.0824	.0847	.0869
4	.0891	.0913	.0936	.0958	.0980	.1003	.1025	.1047	.1069	.1092
5	.1114	.1136	.1159	.1181	.1203	.1225	.1248	.1270	.1292	.1314
6	.1337	.1359	.1381	.1404	.1426	.1448	.1470	.1493	.1515	.1537
7	.1560	.1582	.1604	.1626	.1649	.1671	.1693	.1716	.1738	.1760
8	.1782	.1805	.1827	.1849	.1872	.1894	.1916	.1938	.1961	.1983
9	.2005	.2028	.2050	.2072	.2094	.2117	.2139	.2161	.2183	.2206

Meters to feet.

Meters.	Feet.	Meters.	Feet.	Meters.	Feet.
1=	3.2808	4=	13.1235	7=	22.9661
2=	6.5617	5=	16.4043	8=	26.2470
3=	9.8426	6=	19.6850	9=	29.5278

Feet to meters.

$$1 \text{ foot} = \frac{1}{3.2808} = 0.3048 \text{ meter.}$$

Feet.	Meters.	Feet.	Meters.	Feet.	Meters.
1=	0.3048	4=	1.2192	7=	2.1336
2=	.6096	5=	1.5240	8=	2.4384
3=	.9144	6=	1.8288	9=	2.7432

"Grains per U. S. gallon" to "parts per million."

1 gal.=8.3454 pounds.
1 pound=7.000 grains.
1 gal.=58,418.15 grains.

$$1 \text{ grain per gallon} = \frac{(1)}{(58{,}418.15)} (1{,}000{,}000) = 17.117{,}967 \text{ parts per million.}$$

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	.....	17.1	34.2	51.4	68.5	85.6	102.7	119.8	136.9	154.1
1	171.2	188.3	205.4	222.5	239.6	256.8	273.9	291.0	308.1	325.2
2	342.4	359.5	376.6	393.7	410.8	427.9	445.1	462.2	479.3	496.4
3	513.5	530.6	547.8	564.9	582.0	599.1	616.2	633.4	650.5	667.6
4	684.7	701.8	719.0	736.1	753.2	770.3	787.4	804.5	821.7	838.8
5	855.9	873.0	890.1	907.2	924.4	941.5	958.6	975.7	992.8	1,010.0
6	1,027.1	1,044.2	1,061.3	1,078.4	1,095.5	1,112.7	1,129.8	1,146.9	1,164.0	1,181.1
7	1,198.2	1,215.4	1,232.5	1,249.6	1,266.7	1,283.8	1,301.0	1,318.1	1,335.2	1,352.3
8	1,369.4	1,386.6	1,403.7	1,420.8	1,437.9	1,455.0	1,472.1	1,489.3	1,506.4	1,523.5
9	1,540.6	1,557.7	1,574.8	1,592.0	1,609.1	1,626.2	1,643.3	1,660.4	1,677.6	1,694.7
10	1,711.8	1,728.9	1,746.0	1,763.2	1,780.3	1,797.4	1,814.5	1,831.6	1,848.7	1,865.8

Table of H 3/2 for calculating horsepower of turbines.

Head in feet.	0.0.	.1	.2	.3	.4	.5	.6	.7	.8	.9
0	0.0000	0.0316	0.0894	0.1643	0.2530	0.3536	0.4648	0.5857	0.7155	0 [illegible]
1	1.0000	1.1537	1.3145	1.4822	1.6565	1.8371	2.0238	2.2165	2.4150	2.61[illegible]
2	2.8284	3.0432	3.2631	3.4881	3.7181	3.9529	4.1924	4.4366	4.6853	4.9385
3	5.1962	5.4581	5.7243	5.9947	6.2693	6.5479	6.8305	7.1171	7.4076	7.7019
4	8.0000	8.3019	8.6074	8.9167	9.2295	9.5459	9.8659	10.1894	10.5163	10.[illegible]
5	11.1803	11.5174	11.8578	12.2015	12.5485	12.8986	13.2520	13.6086	13.9682	14.3311
6	14.6969	15.0659	15.4379	15.8129	16.1909	16.5718	16.9557	17.3425	17.7322	18.124[illegible]
7	18.5203	18.9185	19.3196	19.7235	20.1302	20.5396	20.9518	21.3666	21.7842	22.2045
8	22.6274	23.0530	23.4812	23.9121	24.3455	24.7815	25.2202	25.6613	26.1050	26.5513
9	27.0000	27.4512	27.9050	28.3612	28.8199	29.2810	29.7445	30.2105	30.6789	31.1496
10	31.6228	32.0983	32.5762	33.0564	33.5390	34.0239	34.5111	35.0006	35.4924	35.9865
11	36.4829	36.9815	37.4824	37.9855	38.4908	38.9984	39.5082	40.0202	40.5343	41.0506
12	41.5692	42.0910	42.6128	43.1388	43.6648	44.1952	44.7256	45.2600	45.7944	46.3[illegible]
13	46.8720	47.4148	47.9576	48.5048	49.0520	49.6032	50.1544	50.7096	51.2648	51.[illegible]
14	52.3832	52.9464	53.5096	54.0768	54.6440	55.2152	55.7864	56.3616	56.9368	57.5156
15	58.0944	58.6776	59.2608	59.8472	60.4336	61.0244	61.6152	62.2096	62.8040	63.4020
16	64.0000	64.6020	65.2040	65.8096	66.4152	67.0244	67.6336	68.2464	68.8592	69.4760
17	70.0928	70.7132	71.3336	71.9572	75.5808	73.2084	78.8360	74.4672	75.0984	75.7328
18	76.3672	77.0056	77.6440	78.2856	78.9272	79.5724	80.2176	80.8664	81.5152	82.1672
19	82.8192	83.4748	84.1304	84.7892	85.4480	86.1104	86.7728	87.4384	88.1040	88.7732
20	89.4424	90.1152	90.7880	91.4636	92.1392	92.8184	93.4976	94.1800	94.8624	95.5484
21	96.2344	96.9232	97.6120	98.3044	98.9968	99.6924	100.3880	101.0868	101.7856	102.4[illegible]
22	103.1883	103.8940	104.6008	105.3076	106.0160	106.7276	107.4392	108.1540	108.8688	109.[illegible]
23	110.3040	111.0248	111.7456	112.4700	113.1944	113.9216	114.6488	115.3788	116.1088	116.[illegible]
24	117.5752	118.3128	119.0496	119.7876	120.5272	121.2696	122.0120	122.7576	123.5032	124.[illegible]
25	125.0000	125.7516	126.5032	127.2576	128.0120	128.7706	129.5292	130.2876	131.0480	131.[illegible]
26	132.5744	133.3408	134.1072	134.8764	135.6456	136.4180	137.1904	137.9652	138.7400	139.51[illegible]
27	140.2960	141.0768	141.8576	142.6416	143.4256	144.2120	144.9984	145.7880	146.5776	147.37[illegible]
28	148.1624	148.9572	149.7520	150.5500	151.3480	152.1488	152.9496	153.7532	154.5568	155.36[illegible]
29	156.1696	156.9788	157.7880	158.6000	159.4120	160.2268	161.0416	161.8588	162.6760	163.4[illegible]
30	164.3168	165.1396	165.9624	166.7884	167.6144	168.4428	169.2712	170.1020	170.9328	171.76[illegible]
31	172.6008	173.4372	174.2736	175.1128	175.9520	176.7940	177.6360	178.4804	179.3248	180.17[illegible]
32	181.0192	181.8692	182.7192	183.5716	184.4240	185.2792	186.1344	186.9920	187.8496	188.71[illegible]
33	189.5704	190.4336	191.2968	192.1624	193.0280	193.8960	194.7640	195.6348	196.5056	197.37[illegible]
34	198.2520	199.1460	200.0400	200.9008	201.7616	202.6424	203.5232	204.4068	205.2904	206.17[illegible]
35	207.0624	207.9512	208.8400	209.7312	210.6224	211.5204	212.4184	213.3104	214.2024	215.10[illegible]
36	216.0000	216.9012	217.8024	218.7060	219.6096	220.5760	221.4224	222.3312	223.2400	224.15[illegible]
37	225.0624	225.9760	226.8896	227.8056	228.7216	299.6404	230.5592	231.4800	232.4008	233.32[illegible]
38	234.2480	235.1736	236.0992	237.0276	237.9560	238.8868	239.8176	240.7508	241.6840	242.61[illegible]
39	243.5552	244.4932	245.4312	246.3712	247.3112	248.2540	249.1968	250.1420	251.0872	252.05[illegible]
40	252.9624	253.9320	254.8816	255.8340	256.7864	257.7412	258.6960	259.6528	260.6096	261.56[illegible]
41	262.5280	263.4896	264.4512	265.4152	266.3792	267.3456	268.3120	269.2804	270.2488	271.22[illegible]
42	272.1912	273.1644	274.1376	275.1132	276.0888	277.0672	278.0456	279.6252	280.0048	280.98[illegible]
43	281.9696	282.9544	283.9392	284.9264	285.9136	286.9028	287.8920	288.8836	289.8752	290.87[illegible]
44	291.8632	292.8597	293.8552	294.8536	295.8520	296.8528	297.8536	298.8564	299.8592	300.86[illegible]
45	301.8688	302.8764	303.8840	304.8936	305.9032	306.9148	307.9264	308.9404	309.9544	310.97[illegible]
46	311.9872	313.0056	314.0240	315.0448	316.0656	317.0877	318.1112	319.0556	320.0000	321.[illegible]
47	322.2160	323.2452	324.2744	325.3060	326.3376	327.3716	328.4056	329.4416	330.4776	331.51[illegible]
48	332.5536	333.5927	334.6333	335.4753	336.7188	337.7588	338.8051	339.8529	340.8972	341.94[illegible]
49	343.0000	344.0486	345.0986	346.1500	347.2079	348.2622	349.3179	350.3750	351.4336	352.48[illegible]
50	353.5500	354.6128	355.6720	356.7376	357.7996	358.8681	359.9329	360.9992	362.0719	363.14[illegible]

Table of H 3/2 for calculating horsepower of turbines.

Head in feet.	0.0.	.1	.2	.3	.4	.5	.6	.7	.8	.9
51	364.2114	365.2832	366.3564	367.4311	368.5020	369.5794	370.6582	371.7333	372.8149	373.8927
52	374.9772	376.0578	377.1397	378.2331	379.3078	380.3940	381.4815	382.5703	383.6606	384.7522
53	385.8453	386.9343	388.0301	389.1219	390.2205	391.3150	392.4163	393.5136	394.6122	395.7122
54	396.8136	397.9163	399.0204	400.1258	401.2326	402.3408	403.4448	404.5557	405.6679	406.7759
55	407.8855	409.0017	410.1139	411.2273	412.3477	413.4639	414.5814	415.7002	416.8204	417.9419
56	419.0648	420.1833	421.3089	422.4257	423.5583	424.6879	425.8131	426.9458	428.0732	429.2080
57	430.3386	431.4704	432.6086	433.7380	434.8738	436.0110	437.1494	438.2892	439.4302	440.5726
58	441.7106	442.8556	443.9961	445.1438	446.2869	447.4372	448.5830	449.7300	450.8842	452.0359
59	454.0849	455.3271	456.4907	456.6455	457.8017	458.9592	460.1179	461.2720	462.4334	463.5960
60	464.7540	465.9192	467.0797	468.2475	469.4106	470.5750	471.7467	472.9137	474.0819	475.2514
61	476.4222	477.3942	478.7676	479.9422	481.1181	482.2891	483.4676	484.6473	485.8222	487.0044
62	488.1880	489.3666	490.5465	491.7339	492.9163	494.1000	495.2912	496.4774	497.6648	498.8536
63	500.0436	501.2348	502.4273	503.6211	504.8161	506.0061	507.2036	508.4024	509.5961	510.7974
64	512.0000	513.1974	514.3960	515.6024	516.8035	518.0059	519.2160	520.4209	521.6270	522.8344
65	524.0430	525.2528	526.4639	527.6762	528.8898	530.1046	531.3120	532.5313	533.7498	534.9630
66	536.1840	537.2996	538.6230	539.8411	541.0670	542.2875	543.5092	544.7389	545.9630	547.1884
67	548.4151	549.6429	550.8720	552.1022	553.3337	554.5665	555.6179	557.0356	558.2652	559.5027
68	560.7416	561.9748	563.2160	564.4516	565.6953	566.9334	568.1795	569.4199	570.6616	571.9113
69	573.1554	574.4006	575.6473	576.8947	578.1436	579.3937	580.6449	581.8974	583.1510	584.4059
70	585.6620	586.9122	588.1707	589.4303	590.6841	591.9462	593.2023	594.4668	595.7253	596.9921
71	598.2531	599.5152	600.7856	602.0500	603.3157	604.5825	605.8505	607.1197	608.3901	609.6616
72	610.9344	612.2083	613.4340	614.7596	616.0371	617.3085	618.5883	619.8692	621.0841	622.4274
73	623.7120	624.9903	626.2699	627.5579	628.8398	630.1302	631.4144	632.6997	633.9862	635.2813
74	636.5702	637.8602	639.1513	640.4437	641.7372	643.0318	644.3276	645.6246	646.9152	648.2145
75	649.5150	650.8166	652.1118	653.4157	654.7208	656.0195	657.3268	658.6278	659.9375	661.2408
76	662.5452	663.8583	665.1650	666.4728	667.7894	669.0996	670.4108	671.7131	673.0368	674.3514
77	675.6673	676.9842	677.2043	679.6216	680.9419	682.2635	683.5784	684.9021	686.2271	687.5454
78	688.8726	690.2009	691.5226	692.8532	694.1771	695.5100	696.8361	698.1713	699.4997	700.8292
79	702.1599	703.4995	704.8324	706.1665	707.5016	708.8379	710.1752	711.5137	712.8534	714.1941
80	715.5360	716.8789	718.2230	719.5683	720.9146	722.2540	723.6026	724.9523	726.2950	727.6496
81	729.0000	730.3460	731.7613	733.0495	784.3989	785.7575	737.1091	738.4699	739.8237	741.1876
82	742.5346	743.8998	745.2580	746.6173	747.9776	749.3392	750.7018	752.0655	753.4303	754.7962
83	756.1632	757.5312	758.9004	760.2624	761.6338	763.0063	764.3798	765.7461	767.1219	768.4904
84	769.8684	771.2474	772.6192	774.0004	775.3743	776.7493	778.1338	779.5110	780.8892	782.2770
85	783.6575	785.0889	786.4215	787.8052	789.1984	790.5843	791.9712	793.3591	794.7482	796.1383
86	797.5296	798.9219	800.3066	801.7011	803.0966	804.4932	805.8909	807.2810	808.6808	810.0833
87	811.4751	812.8781	814.2736	815.6788	817.0763	818.4837	819.8834	821.2929	822.6947	824.1064
88	825.5704	826.9154	828.3214	829.7374	831.1456	832.5549	833.9652	835.3766	836.7890	838.2025
89	839.6171	841.0327	842.4494	843.8671	845.2859	846.7058	848.1267	849.5487	850.9627	852.3868
90	853.8120	855.2382	856.6564	858.0847	859.5051	860.9355	862.3670	863.7905	865.2241	866.6496
91	868.0763	869.5130	870.9417	872.3806	873.8114	875.2432	876.6761	878.1192	879.5541	880.9901
92	882.4272	883.8652	885.3044	886.7445	888.1857	889.6280	891.0712	892.5156	893.9609	895.4073
93	896.8548	898.3032	899.7528	901.1946	902.6456	904.0982	905.5519	906.9972	908.4530	909.9097
94	911.3582	912.8170	914.2675	915.7284	917.1809	918.6439	920.0985	921.5541	923.0202	924.4778
95	925.9365	927.4056	928.8664	930.3281	931.7908	933.2642	934.7290	936.1948	937.6616	939.1295
96	940.5984	942.0683	943.5392	945.0111	946.4841	947.9581	949.4331	950.9091	952.3764	953.8545
97	955.3336	956.8136	958.2948	959.7672	961.2503	962.7345	964.2099	965.6961	967.1735	968.6617
98	970.1412	971.6314	973.1129	974.6051	976.0886	977.5829	979.0684	980.5548	982.0522	983.5407
99	985.0302	986.5206	988.0220	989.5145	991.0080	992.5025	993.9980	995.4945	996.9920	998.4905
100	1,000.0000	.....	.....	.....	.....	.....	.....	.....	.....	.....

Table of three-halves powers.

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	
.00	0.0000	1.0000	2.8284	5.1962	8.0000	11.1803	14.6969	18.5203	22.6274	27.0000	31.6228	36.4829	.00
.01	0.0010	1.0150	2.8497	5.2222	8.0300	11.2139	14.7337	18.5600	22.6699	27.0450	31.6702	36.5326	.01
.02	0.0028	1.0302	2.8710	5.2482	8.0601	11.2475	14.7705	18.5997	22.7123	27.0899	31.7177	36.5824	.02
.03	0.0052	1.0458	2.8923	5.2743	8.0902	11.2811	14.8073	18.6394	22.7548	27.1351	31.7652	36.6322	.03
.04	0.0080	1.0606	2.9137	5.3004	8.1203	11.3148	14.8442	18.6792	22.7973	27.1802	31.8127	36.6820	.04
.05	0.0112	1.0759	2.9352	5.3266	8.1505	11.3485	14.8810	18.7190	22.8399	27.2253	31.8602	36.7319	.05
.06	0.0147	1.0913	2.9567	5.3528	8.1807	11.3822	14.9179	18.7589	22.8825	27.2705	31.9078	36.7818	.06
.07	0.0185	1.1068	2.9782	5.3791	8.2109	11.4160	14.9549	18.7988	22.9251	27.3156	31.9554	36.8317	.07
.08	0.0226	1.1224	2.9998	5.4054	8.2412	11.4497	14.9919	18.8387	22.9677	27.3608	32.0030	36.8816	.08
.09	0.0270	1.1380	3.0215	5.4317	8.2715	11.4836	15.0289	18.8786	23.0103	27.4060	32.0506	36.9315	.09
.10	0.0316	1.1537	3.0432	5.4581	8.3019	11.5174	15.0659	18.9185	23.0530	27.4512	32.0983	36.9815	.10
.11	0.0365	1.1695	3.0650	5.4845	8.3323	11.5513	15.1030	18.9585	23.0957	27.4965	32.1460	37.0315	.11
.12	0.0416	1.1853	3.0868	5.5110	8.3627	11.5852	15.1400	18.9985	23.1384	27.5418	32.1937	37.0815	.12
.13	0.0469	1.2012	3.1086	5.5375	8.3932	11.6192	15.1772	19.0386	23.1812	27.5871	32.2414	37.1315	.13
.14	0.0524	1.2172	3.1306	5.5641	8.4237	11.6532	15.2143	19.0786	23.2240	27.6324	32.2892	37.1816	.14
.15	0.0581	1.2332	3.1525	5.5907	8.4542	11.6872	15.2515	19.1187	23.2668	27.6778	32.3370	37.2317	.15
.16	0.0640	1.2494	3.1745	5.6173	8.4848	11.7213	15.2887	19.1589	23.3096	27.7232	32.3848	37.2817	.16
.17	0.0701	1.2656	3.1966	5.6440	8.5154	11.7554	15.3260	19.1990	23.3525	27.7686	32.4326	37.3319	.17
.18	0.0764	1.2818	3.2187	5.6708	8.5460	11.7895	15.3632	19.2392	23.3954	27.8140	32.4804	37.3820	.18
.19	0.0828	1.2981	3.2409	5.6975	8.5767	11.8236	15.4005	19.2794	23.4383	27.8595	32.5283	37.4322	.19
.20	0.0894	1.3145	3.2631	5.7243	8.6074	11.8578	15.4379	19.3196	23.4812	27.9050	32.5762	37.4824	.20
.21	0.0962	1.3310	3.2854	5.7512	8.6382	11.8920	15.4752	19.3599	23.5242	27.9514	32.6241	37.5326	.21
.22	0.1032	1.3475	3.3077	5.7781	8.6690	11.9263	15.5126	19.4002	23.5672	27.9960	32.6720	37.5828	.22
.23	0.1103	1.3641	3.3301	5.8050	8.6998	11.9606	15.5501	19.4405	23.6102	28.0416	32.7200	37.6331	.23
.24	0.1176	1.3808	3.3525	5.8320	8.7307	11.9949	15.5866	19.4808	23.6533	28.0872	32.7680	37.6833	.24
.25	0.1250	1.3975	3.3750	5.8590	8.7616	12.0293	15.6250	19.5212	23.6963	28.1328	32.8160	37.7336	.25
.26	0.1326	1.4144	3.3975	5.8861	8.7925	12.0636	15.6616	19.5576	23.7394	28.1784	32.8640	37.7840	.26
.27	0.1403	1.4312	3.4201	5.9132	8.8235	12.0961	15.7001	19.6021	23.7825	28.2241	32.9121	37.8343	.27
.28	0.1482	1.4482	3.4427	5.9403	8.8545	12.1325	15.7376	19.6425	23.8257	28.2698	32.9600	37.8847	.28
.29	0.1562	1.4652	3.4654	5.9675	8.8856	12.1670	15.7752	19.6830	23.8689	28.3155	33.0083	37.9351	.29
.30	0.1643	1.4822	3.4881	5.9947	8.9167	12.2015	15.8129	19.7235	23.9121	28.3612	33.0564	37.9855	.30
.31	0.1726	1.4994	3.5109	6.0220	8.9478	12.2361	15.8505	19.7641	23.9553	28.4069	33.1046	38.0359	.31
.32	0.1810	1.5166	3.5337	6.0493	8.9790	12.2706	15.8882	19.8046	23.9986	28.4527	33.1527	38.0864	.32
.33	0.1896	1.5338	3.5566	6.0767	9.0102	12.3053	15.9260	19.8452	24.0418	28.4985	33.2009	38.1369	.33
.34	0.1983	1.5512	3.5795	6.1041	9.0414	12.3399	15.9637	19.8858	24.0851	28.5444	33.2492	38.1874	.34
.35	0.2071	1.5686	3.6025	6.1315	9.0726	12.3746	16.0015	19.9265	24.1285	28.5902	33.2974	38.2379	.35
.36	0.2160	1.5860	3.6255	6.1590	9.1040	12.4093	16.0393	19.9672	24.1718	28.6361	33.3457	38.2884	.36
.37	0.2251	1.6035	3.6486	6.1865	9.1353	12.4440	16.0772	20.0079	24.2152	28.6820	33.3940	38.3390	.37
.38	0.2342	1.6211	3.6717	6.2141	9.1667	12.4788	16.1150	20.0486	24.2586	28.7279	33.4423	38.3896	.38
.39	0.2436	1.6388	3.6949	6.2417	9.1981	12.5136	16.1529	20.0894	24.3021	28.7739	33.4906	38.4402	.39
.40	0.2530	1.6565	3.7181	6.2693	9.2295	12.5485	16.1909	20.1302	24.3455	28.8199	33.5390	38.4908	.40
.41	0.2625	1.6743	3.7413	6.2970	9.2610	12.5833	16.2288	20.1710	24.3890	28.1659	33.5874	38.5415	.41
.42	0.2722	1.6921	3.7646	6.3247	9.2925	12.6182	16.2668	20.2118	24.4325	28.9119	33.6358	38.5922	.42
.43	0.2820	1.7100	3.7880	6.3525	9.3241	12.6532	16.3048	20.2527	24.4761	28.9579	33.6842	38.6429	.43
.44	0.2919	1.7280	3.8114	6.3803	9.3557	12.6882	16.3429	20.2936	24.5196	29.0040	33.7327	38.6936	.44
.45	0.3019	1.7460	3.8349	6.4081	9.3873	12.7232	16.3810	20.3345	24.5632	29.0501	33.7811	38.7443	.45
.46	0.3120	1.7641	3.8584	6.4360	9.4189	12.7582	16.4191	20.3755	24.6068	29.0962	33.8297	38.7951	.46
.47	0.3222	1.7823	3.8819	6.4639	9.4506	12.7933	16.4572	20.4165	24.6505	29.1424	33.8782	38.8459	.47
.48	0.3325	1.8005	3.9055	6.4919	9.4824	12.8284	16.4954	20.4575	24.6941	29.1885	33.9267	38.8967	.48
.49	0.3430	1.8188	3.9292	6.5199	9.5141	12.8635	16.5336	20.4985	24.7378	29.2347	33.9753	38.9475	.49
.50	0.3536	1.8371	3.9529	6.5479	9.5459	12.8986	16.5718	20.5396	24.7815	29.2810	34.0239	38.9984	.50

Table of three-halves powers—Continued.

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	
.51	0.3642	1.8555	3.9766	6.5760	9.5778	12.9338	16.6101	20.5807	24.8253	29.3272	34.0725	39.0493	.51
.52	0.3750	1.8740	4.0004	6.6041	9.6097	12.9691	16.6484	20.6218	24.8691	29.3735	34.1211	39.1002	.52
.53	0.3858	1.8925	4.0242	6.6323	9.6416	13.0043	16.6867	20.6630	24.9129	29.4198	34.1698	39.1511	.53
.54	0.3968	1.9111	4.0481	6.6605	9.6735	13.0396	16.7250	20.7041	24.9567	29.4661	34.2185	39.2020	.54
.55	0.4079	1.9297	4.2520	6.6887	9.7055	13.0749	16.7634	20.7453	25.0005	29.5124	34.2672	39.2530	.55
.56	0.4191	1.9484	4.0960	6.7170	9.7375	13.1103	16.8018	20.7866	25.0444	29.5588	34.3159	39.3040	.56
.57	0.4303	1.9672	4.1200	6.7453	9.7695	13.1457	16.8402	20.8278	25.0883	29.6052	34.3647	39.3550	.57
.58	0.4417	1.9860	4.1441	6.7737	9.8016	13.1811	16.8787	20.8691	25.1322	29.6516	34.4135	39.4060	.58
.59	0.4532	2.0049	4.1682	6.8021	9.8337	13.2165	16.9172	20.9104	25.1762	29.6980	34.4623	39.4571	.59
.60	0.4648	2.0238	4.1924	6.8305	9.8659	13.2520	16.9557	20.9518	25.2202	29.7445	34.5111	39.5082	.60
.61	0.4764	2.0429	4.2166	6.8590	9.8981	13.2875	16.9943	20.9931	25.2642	29.7910	34.5599	39.5593	.61
.62	0.4882	2.0619	4.2408	6.8875	9.9303	13.3231	17.0328	21.0345	25.3082	29.8375	34.6088	39.6104	.62
.63	0.5000	2.0810	4.2651	6.9161	9.9626	13.3587	17.0714	21.0759	25.3522	29.8841	34.6577	39.6615	.63
.64	0.5120	2.1002	4.2895	6.9447	9.9949	13.3943	17.1101	21.1174	25.3963	29.9306	34.7066	39.7127	.64
.65	0.5240	2.1195	4.3139	6.9733	10.0272	13.4299	17.1488	21.1589	25.4404	29.9772	34.7557	39.7639	.65
.66	0.5362	2.1388	4.3383	7.0020	10.0596	13.4166	17.1874	21.2004	25.4845	30.0238	34.8045	39.8151	.66
.67	0.5484	2.1581	4.3628	7.0307	10.5920	13.5013	17.2172	21.2419	25.5287	30.0704	34.8535	39.8663	.67
.68	0.5607	2.1775	4.3874	7.0595	10.1244	13.5370	17.2649	21.2834	25.5729	30.1171	34.9025	39.9176	.68
.69	0.5732	2.1970	4.4119	7.0883	10.1569	18.5728	17.3037	21.3250	25.6171	30.1638	34.9516	39 9689	.69
.70	0.5857	2.2165	4.4366	7.1171	10.1894	13.6086	17.3425	21.3666	25.6613	30.2105	35.0006	40.0202	.70
.71	0.5983	2.2361	4.4612	7.1460	10.2214	13.6444	17.3814	21.4083	25.7056	30.2572	35.0497	40.0715	.71
.72	0.6109	2.2558	4.4859	7.1749	10.2545	13.6803	17.4202	21.4499	25.7499	30.3040	35.0988	40.1228	.72
.73	0.6237	2.2755	4.5107	7.2038	10.2871	13.7161	17.4591	21.4916	25.7942	30.3507	35.1479	40.1742	.73
.74	0.6366	2.2952	4.5355	7.2328	10.3197	13.7521	17.4981	21.5333	25.8395	30.3975	35.1971	40.2256	.74
.75	0.6495	2.3150	4.5604	7.2618	10.3524	13.7880	17.5370	21.5751	25.8828	30.4444	35.2462	40.2770	.75
.76	0.6626	2.3349	4.5853	7.2909	10.3851	13.8240	17.5760	21.6169	25.9272	30.4912	35.2954	40.3284	.76
.77	0.6757	2.3548	4.6102	7.3200	10.4178	13.8600	17.6150	21.6587	25.9716	30.5381	35.3446	40.3798	.77
.78	0.6889	2.3748	.6352	7.3492	10.4506	13.8961	17.6541	21.7005	26.0161	30.5850	35.3939	40.4313	.78
.79	0.7022	2.3949	4.6602	7.3783	10.4834	13.9321	17.6931	21.7423	26.0605	30.6319	35.4431	40.4828	.79
.80	0.7155	2.4150	4.6853	7.4076	10.5163	13.9682	17.7322	21.7842	26.1050	30.6789	35.4924	40.5343	.80
.81	0.7290	2.4351	4.7104	7.4368	10.5492	14.0044	17.7714	21.8261	26.1495	30.7258	35.5417	40.5859	.81
.82	0.7425	2.4553	4.7356	7.4661	10.5812	14.0406	17.8105	21.8681	26.1941	30.7728	35.5911	40.6374	.82
.83	0.7562	2.4756	4.7608	7.4955	10.6150	14.0768	17.8567	21.9100	26.2386	30.8198	35.6404	40.6890	.83
.84	0.7699	2.4959	4.7861	7.5248	10.6480	14.1130	17.8889	21.9520	26.2832	30.8669	35.6898	40.7406	.84
.85	0.7837	2.5163	4.8114	7.5542	10.6810	14.1493	17.9282	21.9940	26.3278	30.9139	35.7392	40.7922	.85
.86	0.7975	2.5367	4.8367	7.5837	10.7141	14.1856	17.9674	22.0361	26.3725	30.9610	35.7886	40.8439	.86
.87	0.8115	2.5572	4.8621	7.6132	10.7472	14.2219	18.0067	22.0781	26.4171	31.0081	35.8380	40.8955	.87
.88	0.8255	2.5777	4.8875	7.6427	10.7803	14.2582	18.0461	22.1202	26.4618	31.0553	35.8875	40.9472	.88
.89	0.8396	2.5983	4.9130	7.6723	10.8134	14.2946	18.0854	22.1623	26.5065	31.1024	35.9370	40.9989	.89
.90	0.8538	2.6190	4.9385	7.7019	10.8466	14.3311	18.1248	22.2045	26.5523	31.1496	35.9865	41.0507	.90
.91	0.8681	2.6397	4.9641	7.7315	10.8798	14.3675	18.1642	22.2467	26.5960	31.1968	36.0360	41.1024	.91
.92	0.8824	2.6604	4.9897	7.7702	10.9131	14.4040	18.2037	22.2889	26.6408	31.2441	36.0856	41.1542	.92
.93	0.8969	2.6812	5.0154	7.7909	10.9464	14.4405	18.2432	22.3311	26.6856	31.2913	36.1352	41.2060	.93
.94	0.9114	2.7021	5.0411	7.8207	10.9797	14.4770	18.2827	22.3733	26.7305	31.3386	36.1848	41.2578	.94
.95	0.9259	2.7230	5.0668	7.8505	11.0131	14.5136	18.3222	22.4156	26.7753	31.3850	36.2344	41.3097	.95
.96	0.9406	2.7440	5.0926	7.8803	11.0464	14.5502	18.3617	22.4579	26.8202	31.4332	36.2841	41.3615	.96
.97	0.9553	2.7650	5.1184	7.9102	11.0799	14.5869	18.4013	22.5003	26.8651	31.4806	36.3337	41.4134	.97
.98	0.9702	2.7861	5.1443	7.9401	11.1133	14.6235	18.4409	22.5426	26.9100	31.5280	36.3834	41.4653	.98
.99	0.9850	2.8072	5.1702	7.9700	11.1468	14.6602	18.4806	22.5850	26.9550	31.5754	36.4331	41.5173	.99
1.00	1.0000	2.8284	5.1962	8.0000	11.1803	14.6969	18.5203	22.6274	27.0000	31.6228	36.4829	41.5692	1.00

CONVENIENT EQUIVALENTS.

1 second-foot equals 50 California miner's inches.
1 second-foot equals 38.4 Colorado miner's inches.
1 second-foot equals 40 Arizona miner's inches.
1 second-foot equals 7.48 United States gallons per second.
1 second-foot equals 6.23 British imperial gallons.
1 second-foot for one day equals 1.9835 acre-feet.
1 second-foot for one day equals 646,272 United States gallons.
1 second-foot for one year equals 0.000214 cubic mile.
1 second-foot for one year covers 1 square mile 1.131 feet deep.
1 second-foot equals 449.9 gallons per minute.
1 second-foot equals about one acre-inch per hour.
1 cubic foot of water weighs 62.47 pounds.
100 California miner's inch equals 2 second-feet.
100 California miner's inches equals 15 United States gallons per second.
100 California miner's inches equals 77 Colorado miner's inches.
100 California miner's inches for one day equals 4 acre-feet.
100 Colorado miner's inches equals 2.60 square feet.
100 Colorado miner's inches equals 19.5 United States gallons per second.
100 Colorado miner's inches equals 130 California miner's inches.
100 Colorado miner's inches for one day equals 5.2 acre-feet.
100 United States gallons per minute equals .223 second-foot.
100 United States gallons per minute for one day equals 44 acre-feet.
1 million United States gallons per day equals 1.55 second-feet.
1 million United States gallons equals 3.07 acre-feet.
1 million cubic feet equals 22.95 acre-feet.
1 acre-foot equals 325,850 gallons.
A layer 1 inch deep on one square mile equals 2,323,200 cubic feet.
A flow of 1 second-foot in one year equals 31,536,000 cubic feet.
1 inch deep on 1 square mile equals 0.0737 second-foot per year.
10 inches deep on 1 square mile equals 0.7367 second-foot per year.
A flow of 1 second-foot per year covers 1 square inch 13.589.
1 cubic mile equals 147,198,000,000 cubic feet.
1 cubic mile equals 4,667 second-feet.
1 second-foot per year equals 31,536,000 cubic feet.
1 second-foot per year equals 0.000214 cubic mile.
1 foot per second equals 1.077 kilometers per hour.
1 foot per second equals 0.68 mile per hour.
1 inch equals 2.54 centimeters.
1 foot equals 0.3048 meter.
1 yard equals 0.9144 meters.
1 mile equals 1.60935 kilometers.
1 square yard equals 0.836 square meter.
1 acre equals 0.4047 hectare.
1 square mile equals 259 hectares.
1 square mile equals 2.59 square kilometers.
1 cubic foot equals 0.0283 cubic meter.
1 cubic yard equals 0.7646 cubic meter.
1 gallon equals 3.7854 liters.
1 pound equals 0.4536 kilogram.
1 atmosphere equals about 15 pounds per square inch, 1 ton per square foot, 1 kilo per square centimeter.
Acceleration of gravity equals 32.16 feet per second every second.

1 acre equals 209 feet square, nearly.
1 acre equals 43,560 square feet, equals 4,840 square yards.
1 mile equals 1,760 yards, equals 5,280 feet, equals 63,360 inches.
1 cubic foot equals 7.48 gallons, equals 0.804 bushel.
1 gallon equals 8.34 pounds of water.
1 gallon equals 231 cubic inches (liquid measure).
1 avoirdupois pound equals 7,000 grains.
1 troy pound equals 5,760 grams.
1 meter equals 39.37 inches. Log. 1.5951654.
1 meter equals 3.28083 feet. Log. 0.5159842.
1 meter equals 1.093611 yards. Log. 0.0388629.
1 meter equals 0.00062137 mile. Log. 6.7933495.
1 kilometer equals 3,281 feet, equals $\frac{5}{8}$ mile, nearly.
1 square meter equals 10,764 square feet, equals 1.196 square yard.
1 hectare equals 2.471 acres.
1 cubic meter equals 35.314 cubic feet, equals 1.308 cubic yards.
1 liter equals 1.0567 quarts.
1 gram equals 15.43 grains.
1 kilogram equals 2.2046 pounds.
1 tonneau equals 2,204.6 pounds.
1 cubic meter per minute equals 0.5886 second-foot.
1 horsepower equals 550 foot-pounds per second.
1 horsepower equals 76 kilogrameters per second.
1 horsepower equals 746 watts.
1 horsepower equals 1 second-foot of water falling 8.8 feet.
1 second-foot falling 10 feet equals 1.135 horsepower.
$1\frac{1}{3}$ horsepowers equals about 1 kilowatt.

To calculate water power quickly: $\frac{\text{Sec.-ft. x fall in feet.}}{11} =$ Net horsepower on water wheel, realizing 80 per cent of the theoretical power.

Quick formula for computing discharges over weirs: Cubic feet per minute equals $0.4025\ l\sqrt{h^3}$; l=length of weir in inches; h=head in inches flowing over weir, measured from surface of still water.

To change miles to inches on map:

Scale 1:125000,	1 mile = 0.50688 inches.	Log. = 9.7049052.
Scale 1:90000,	1 mile = 0.70400 inches.	Log. = 9.8475727.
Scale 1:62500,	1 mile = 1.01376 inches.	Log. = 0.0059352.
Scale 1:45000,	1 mile = 1.40800 inches.	Log. = 0.1486027.

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DEPARTMENT OF THE INTERIOR
UNITED STATES GEOLOGICAL SURVEY
CHARLES D. WALCOTT, DIRECTOR

ACCURACY

OF

STREAM MEASUREMENTS

(SECOND, ENLARGED EDITION)

BY

EDWARD CHARLES MURPHY

WASHINGTON
GOVERNMENT PRINTING OFFICE
1904

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
HYDROGRAPHIC BRANCH,
Washington, D. C., December 5, 1903.

SIR: I have the honor to transmit herewith a manuscript entitled "Accuracy of Stream Measurement," by Edward C. Murphy.

This paper is a revision and enlargement of paper No. 64. It embodies the results of an extension of the investigations, in the hydraulic laboratory of Cornell University, of the flow of small and moderate-sized streams.

While the methods of river measurement at present in use are believed to be sufficiently accurate for the purposes of ascertaining water resources, yet it is important to know their probable accuracy and particularly the accuracy of the instruments which are commonly employed. Through such knowledge and experience gained from time to time it will be possible to improve upon the methods, in the interest of greater exactness or wider diffusion of the work. I therefore request that this manuscript be published in the series of Water-Supply and Irrigation Papers.

Very respectfully,

F. H. NEWELL,
Chief Engineer.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

ACCURACY OF STREAM MEASUREMENTS.

By EDWARD C. MURPHY.

INTRODUCTION.

Since 1902, when the first edition of the paper on the accuracy of stream measurements was published, many additional data relating to this subject have been collected and studied, both by the author and by various engineers of the United States Geological Survey.

In order to give the public the benefit of these later data and studies, and also to supply the continued demand for the first edition, the supply of which had become exhausted, it was thought desirable to prepare and issue a second edition. In preparing this enlarged edition the material presented in the first edition has been revised in accordance with the new data.

FACTORS CONTROLLING ACCURACY OF STREAM MEASUREMENTS.

The accuracy of a stream measurement depends largely upon the accuracy with which the cross-sectional area and the velocity are measured. There is no special difficulty in measuring the first factor, but the second factor—the velocity—is very difficult to measure, chiefly for the reason that it is constantly changing. It not only varies from the surface to the bottom and from one bank of the stream to the other, so that it is necessary to measure it at many points, but it is constantly changing at every point, even when the cross-sectional area and the discharge (and consequently the mean velocity) remain constant. Several experimenters have observed the phenomenon of "pulsation of moving water," and a few have tried to measure it, but as yet little is known of the magnitude and frequency of the pulsations or of the laws governing them. J. B. Francis says:[a] "It is observed that there is a continual change in all parts of these channels, although there may be no sensible change in the volume of water flowing and conse-

[a] Trans. Am. Soc. Civ. Eng., vol. 7, p. 111.

quently in the mean velocity." Captain Cunningham says:[a] "One of the most important conclusions of modern experiments is that the motion of water, even when tranquil to the eye, is extremely unsteady, so that there is no definite velocity at any point, but the velocity varies everywhere, largely from instant to instant. * * * It is analogous to the unsteady motion of wind, which is exemplified by the swaying of wind vanes and by the fluttering of pinions." D. F. Henry says:[b] "All water in motion has an intermittent velocity, increasing and decreasing according to some undiscovered law," and Henry found this true of streams of all sizes, from small mill races to the great St. Lawrence River.

A knowledge of these phenomena is evidently of vital importance in making and computing stream measurements. If only a few observations of velocity are made, these may all, or nearly all, be made at a time of maximum impulse, and thus the measured mean velocity be too large; or it is possible that most of the observations may be made at a time of minimum impulse, and thus the mean velocity be too small. They also have an important bearing on the kind of instrument best suited to measure the velocity, as some—the float rod, for example—give the velocity of a single impulse, while others, as the current meter, show the average velocity due to all the impulses during the observation.

The motion of water in an open channel is not, however, simply a succession of impulses. On the contrary, it is exceedingly complex, very different from the uniform flow in parallel straight lines that is assumed in deriving the common hydraulic formulæ. When closely observed the water of the most undisturbed streams is seen to "boil" and swirl and to be very unsteady, some particles moving up, others down, others across, but all as a rule having a general motion downstream.

The accuracy of a discharge measurement also depends much upon the physical features of the stream at the discharge section or point of measurement. When possible, this section should be on a straight reach and far enough from a bend to be out of its influence, the bed should be permanent and not stony, and the slope and wetted perimeter such that at high and low stages of the stream the velocity in all parts of the section will be easily measureable. The banks should be sufficiently high not to be overflowed at flood stage, and the section should be free from the influence of milldams and bridge piers. In addition, economy requires that the section selected be easily accessible from a railway station and that there be a person living near who will read the elevation of the water surface at stated times. Seldom if ever are all of these conditions even approximately satisfied. Scour

[a] Recent Hydraulic Experiments: Proc. Inst. Civ. Eng., vol. 71, p. 7.
[b] Jour. Franklin Inst., vol. 62, p. 323.

of bed in some cases and silting in other cases give much trouble; too great a velocity at high stages and too low a velocity at low stages to measure with accuracy are common difficulties; and milldams often give trouble, on account of storage and irregular discharge through their wheels; so that the hydrographer must make the most of the best location he can find, always having in mind the desirable and the undesirable qualities.

Rapid fluctuations of the water surface or river height during measurement and the condition of the velocity-measuring instrument are other factors which affect the accuracy of a stream measurement.

A great many discharge measurements of natural and artificial channels have been made with various kinds of instruments and in various ways. The earlier ones were made with crude instruments, and in some cases the surface velocity only was observed, the mean velocity being computed from a formula which we now know is not correct. Very little appears to have been done in the way of determining the degree of accuracy of the measurements. Even when the experimenter has used two or more instruments to measure velocity, he does not appear to have made simultaneous measurements with different instruments, or employed different methods with the same instruments, in order to test the accuracy of the results.

These matters are all discussed in detail in the following pages.

METHODS OF MEASURING VELOCITY.

The methods that have been used for measuring velocity may be divided into two general classes—the direct and indirect. The direct methods include all ways of ascertaining the velocity of water from bodies floating in it, such as surface floats, float rods, double floats, etc. The indirect methods include measurements with the current meter, pressure plates, the thermometer, etc., by which the velocity of the water is inferred from its impact, its pressure, or its temperature. Measurements by the direct methods are simple, requiring only the measurement of the time that it takes a body in the water to move over a certain distance, while in the indirect methods the constant of the instant must first be found from experiment. The latter methods are, however, theoretically better than the former.

By the direct methods the mean velocity of a comparatively few particles of water is found for the time required for the float to move between the sections of observation—that is, the float is acted upon by the particles about it during the time it is passing over the measured distance. It is carried along by a single impulse of the water, and is not affected by the succeeding impulses, which may be greater or less than the impulse which moves it. Instruments of the second class, on the other hand—as, for instance, the meter—measure instan-

taneously the velocity of all the particles that strike it during the observation. If there were no pulsations of water, and if the particles following one another at any given point moved with uniform velocity, then the direct methods might, in the absence of wind, give good results; but since all moving water, even the most undisturbed, has these pulsations, velocity measurements by the indirect methods are decidedly more accurate than those by the direct methods. One measurement with a current meter is worth several measurements with floats.

There is another reason why indirect measurements are preferable to direct measurements. Since in the latter the mean velocity is found over a certain distance, usually from 50 to 200 feet, it is necessary to know the mean area over that distance, and this requires the measurement of many cross sections between the upper and the lower section, while by the indirect methods the measurement of only one area is required.

DIRECT METHODS.

SURFACE FLOATS.

Surface floats are small, light bodies, such as wood or wax, which float on the surface of the water and can readily be seen from the shore. Measurement by this means is rapid, but the velocities obtained may be greatly in error, due to the action of wind on the float. At best it only indicates the velocity for a short time of a comparatively few particles of water, and the mean velocity must be found from the relation between surface velocity and mean velocity, which is not well established. The time consumed by the floats in passing over a measured distance—usually from 50 to 200 feet—is observed, and the distance divided by the observed time gives the surface velocity at that point for that time.

This method was used by Dubuat in 1779 in gaging the Canal de Jard, in France, and in 1782 in gaging the Haine River, Belgium; by Trechsel, in 1825, to gage the Outer Aar, near Thun; by Wampfler, in 1867, to gage the Simme Canal, Switzerland; by Harlacher, in 1881, to gage the Elbe, in Bohemia (at high water only); by Ellet, in 1837, in gaging the Ohio; and by others.

DOUBLE FLOATS.

The double float consists of a light surface float and a subsurface float somewhat heavier than water connected to it by a cord or a small rope. The office of the upper float is to support the lower float and indicate its position. The connecting cord can be lengthened at will and the lower float be placed at any desired depth. Fig. 1 shows the double float used by T. G. Ellis in the Connecticut River survey in

1874. This subsurface float was a hollow annulus of tin 8½ inches high, 8½ inches outside diameter, and 7½ inches inside diameter. Two brass wires were soldered across the bottom at right angles to each other, to which was attached 28 ounces of lead as a sinker. Two other wires were soldered at right angles to each other at the center, to which the connecting cord was attached. The surface float was an ellipsoid of tin 6 inches in diameter and 1½ inches thick, with a cork in the top holding a small flag and an eye in the bottom for the connecting cord. The connecting cord had a diameter of 0.036 inch.

The method of measuring stream flow by double floats as used by Humphreys and Abbot in gaging the Mississippi River in 1851 and 1858 is substantially as follows:[a] A suitable place was selected on the river, a base line 200 feet long was measured off on one bank parallel to the axis of the current, and a section at each end of this base was marked out at right angles to it. The time consumed by each float in passing between these sections and the position of each float when it passed them were noted by four men, two at each end of the base, each party having a stop watch and a theodolite. At a signal from the engineer the floats were placed in the river from a boat about 100 feet above the upper section and were picked up by a man in a boat some distance below the lower section. At the instant a float passed the upper section a signal was given, the watches were started, and the angular position of the float was read with both theodolites. The instant the float passed the lower section a signal was again given, the watches were stopped, and the angular position of the float was again read with both theodolites. These readings gave the distance of the float from the base line when it passed the sections and two independent measurements of the time consumed by it in passing between the sections. Soundings were taken at the end sections and at one or more intermediate sections, from which the area of the mean section was computed.

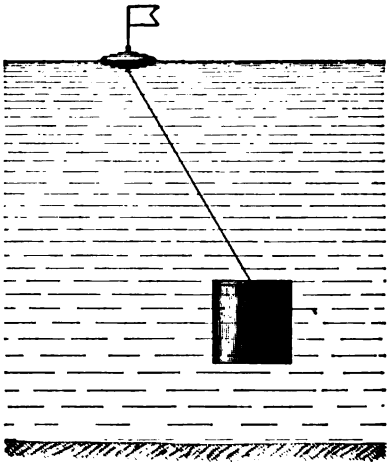

FIG. 1.—Double float used by Ellis in Connecticut River survey in 1874.

In some cases where this method has been used the base was 300 feet long and the time of run of the floats has been recorded on a chronograph.

This method has its advantages and its disadvantages. In very deep rivers, such as the Mississippi, or in streams carrying weeds

[a] Report on the Mississippi River, by Humphreys and Abbot, p. 224.

and grass, it is almost the only available method. Humphreys and Abbot, in their report on the Mississippi River, say:[a] "Saxton's current meter was tried but found to be unsuited to measurements in a river of such great depth and violence of current. Only double floats were found to give reliable results."

In regard to the disadvantages, D. F. Henry, who had charge of the field work of the gaging of the outlets of the Great Lakes and who has used this and other methods for deep rivers, says:[b] "All the objections to the surface float apply with greater force to the double float, and additional ones peculiar to itself." It is impossible to determine the exact position or depth of the lower float. Its position is determined from that of the surface float, but it varies with the direction and velocity of the wind and the length of the cord connecting the floats. The depth of the float is determined from the length of the connecting cord, but on account of the upward "boiling" motion of the water, and also the pressure of the water on a long connecting cord, the depth may be much less than the length of the cord. The upper float may drag the lower one or be dragged by it. At best it can only give the velocity of a few particles for a short space of time.

This method of gaging was used in Mississippi River and tributaries from 1857 to 1881. Since that time the current meter has been largely used. In 1869 it was used in conjunction with the current meter for gaging the outlets of the Great Lakes. It was also used by T. G. Ellis in 1874 in his survey of the Connecticut River, in conjunction with the current meter, and by Gordon in 1873 to gage the Irawadi River. For comparisons of accuracy of this method with others, see pages 48 to 52.

FLOAT RODS.

Professor Cabeo[c] was the first to use the float rod for measuring velocity. This was in 1646. The rod is of wood or tin, from 1 to 2 inches in diameter, weighted at the lower end so as to float vertically. Its lower end should nearly touch the bottom and its upper end project a few inches above the surface of the water, so as to be visible. The method of using float rods as followed by the writer in 1900 in the New York State canal survey is as follows: Two surveyors' 100-foot chains were stretched across the canal from 10 to 30 feet apart and at right angles to the axis of the canal. Fifteen feet above the upper chain a rope was stretched across the canal, and to this a boat was attached by pulleys, so that a man in the boat could easily move himself back and forth by pulling on the rope. Another rope was

[a] Op. cit., p. 225.
[b] Jour. Franklin Inst., vol. 62, p. 167.
[c] See Report of New York Barge Canal, 1901, p. 869.

stretched across the canal from 6 to 8 feet below the lower chain and a boat operated from it in a similar way. The floats were put in by the man in the upper boat, two at a time and about five feet apart, and were observed by him until they passed the upper chain, when he called to the recorder the position of each. As they passed under the lower chain the man in the lower boat called out their position, and when they reached his boat he took them out of the water and brought them to the shore. An observer with two stop watches noted the time of passage of each pair of floats between the chains. A short run was used on account of the low velocity of the water.

The advantages and disadvantages of the method may be summed up briefly as follows:

Advantages.—For artificial channels of moderate and uniform depth and with floating grass and weeds this is probably the best method to use. The advantages of float rods as given by Captain Cunningham, who has used them to a large extent in his gagings of the Ganges Canal, are as follows:[b] (1) They interfere less with the natural motion of the water; (2) they measure velocity direct; (3) they can be used in a stream of any size; (4) they are not affected by silt and weeds; (5) they measure forward velocity; (6) they can be made by a common workman; and (7) they are cheap.

Disadvantages.—The rods are affected to some extent by wind; they do not give mean velocity, but velocity of impulses; they can not be used in deep streams or in streams of rough or irregular bed, and they are expensive to operate.

Float rods have been used by Cunningham, on the Ganges Canal, in 1880; by Francis, in 1852, on a canal at Lowell, Mass.; by Buffon, on the Tiber River, in 1821; by Krayenhoff, on the Rhine, in 1812; and by the Mississippi River Commission, on the Upper Mississippi, in 1881.

FLOAT FRAME.

Hirn used, for obtaining velocity, a light frame of wood which nearly filled the cross section of the channel, but there are practical difficulties in the way of the use of this instrument.

INDIRECT METHODS.

The following instruments and methods have been used at one time or another for measuring velocity indirectly. Several of them are now of historical interest only.

FLOAT WHEEL.

The float wheel was used by Borda and Dubuat for measuring surface velocities. That used by the latter was 2 feet in diameter, and recorded the revolutions on dials.

[b] Proc. Inst. Civil Eng., vol. 71, p. 1.

PRESSURE PLATE.

In 1779 Gaunthy invented the pressure plate, which consists of a disk of metal opposed to the pressure of water, the velocity being computed from the weight necessary to keep it vertical. The tachometer used by Brünings to gage the Rhine and the one used by Racourt in his gagings of the Neva were made on this principle. Captain Boileau also used a pressure plate in his tachometer.

BOX WITH HOLE IN SIDE.

A box with a hole in the side was used by Grandi in 1730 to measure velocity. It was lowered gradually from the surface to the bottom and raised again at the same rate, the velocity being inferred from the amount of water in the box.

HYDROMETRIC TUBE.

This instrument was used by Captain Boileau in 1850. It consists of a glass tube suspended in a frame, having a full-sized opening at one end and a small opening at the other end. The tube is filled with water and a bubble of air, and is placed in the water with the small end upstream. The large end is then opened and the time required for the bubble to traverse the tube is noted. From this time and the areas of the ends of the tube the velocity is computed.

HYDROMETRIC PENDULUM.

This instrument was used by Castelli in 1628. It consists of a ball suspended from the center of a graduated arc. The velocity is computed from the weight of the ball and the angle of the string when the water impinges on it.

THERMOMETER.

Leslie used a thermometer for measuring velocity. The principle of this method is that the temperature of water in motion is greater than the temperature when at rest.

PITOT TUBE.

This instrument was invented by Pitot in 1730. Fig. 2 shows it as improved and used by Darcy and Bazin.[a] It consists essentially of two tubes, one drawn to a fine point and pointing upstream, the other straight and with an opening at the lower end the size of the tube. The air in both tubes is partly exhausted, so that the water columns

[a] Recherches hydrauliques, entreprises par M. H. Darcy continuées par M. H. Bazin. Première partie de Recherches expérimentales sur l'écoulement de l'eau dans les canaux découverts: Extrait des Mémoires présentés par divers savants à l'académie des sciences de l'Institut Impérial de France. Paris, 1865, vol. 19, pl. xvii, fig. 5.

will rise to a convenient height. The velocity is found from the difference in the readings of the water columns. Like floats, this is an impulse-measuring instrument, but in using it both maximum and minimum impulses are observed and a mean is found. After the instrument is in place and the stopcocks R and R' are opened one of the observers applies his mouth to the tube O, removes the air until the surface of the water in the tubes stands at a convenient height to read, and then closes the upper cock (R). He observes the columns until they reach a maximum height, when he closes the cock R' and reads the surface of both columns; then he opens the cock R and observes the surface of both until the columns reach a minimum position, when he closes cock R' and reads both surfaces again. He now has a maximum and a minimum reading of each column, from which he makes the following deduction: If a' is the maximum reading of column A, and b' is the maximum reading of column B, and a'' is the minimum reading of column A, and b'' is the minimum reading of column B, then $a'-b'=d'$, the difference for the maximum impulse, and $a''-b''=d''$, the difference for the minimum impulse, and $\frac{d' \times d''}{2}$ is the mean difference of the column readings for the two impulses.

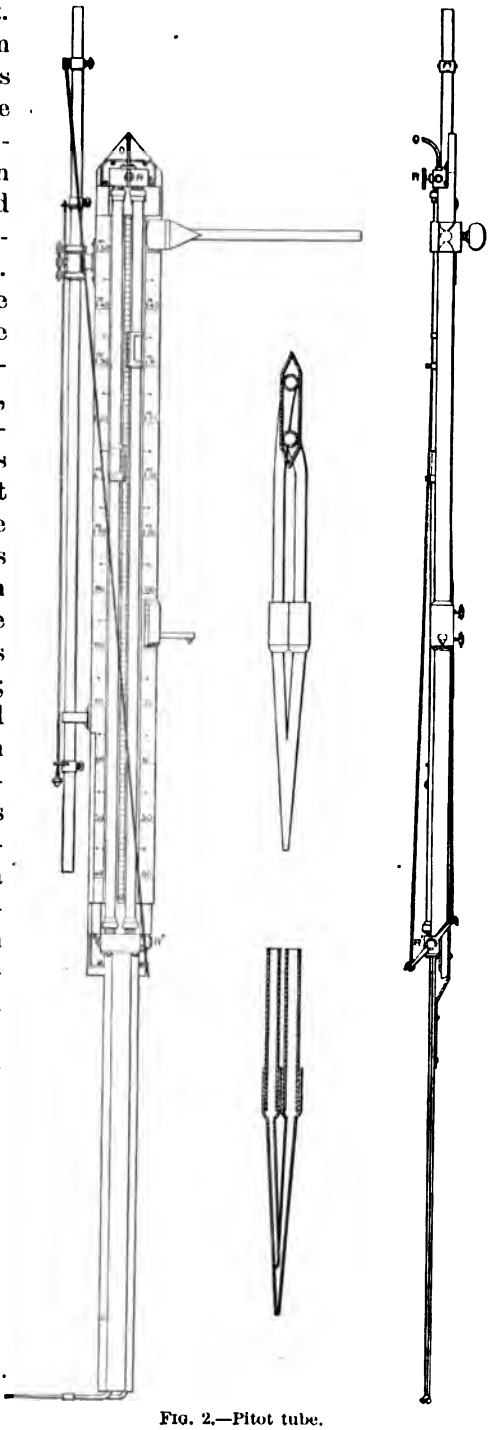

FIG. 2.—Pitot tube.

Two or three sets of these readings are taken at each point. Then if $\Sigma\frac{d}{2}$ is the mean of the values of these column differences ($\Sigma d = d'+d''+ \ . \ . \ .$), the velocity at that point is given by the formula $V=c\sqrt{2g\Sigma\frac{d}{2}}$, c being the coefficient found by rating and g the acceleration of gravity. The velocity can very quickly be found by one accustomed to using the instrument. The accuracy of V depends on that of c and of $\sqrt{\Sigma d}$, the greater the number of d's or sets of observations, the more nearly accurate will be the value of V.

CURRENT METER.

The current meter had its beginning in the float wheel used by Borda and Dubuat to obtain surface velocities. In 1790 Waltman modified this wheel so that it could be used below the surface. His meter had helicoidal blades and an endless screw on its axis, which by gearing into a train of wheels caused a record of the number of revolutions of the wheel to be made on dials. It was moved up and down on a rod, the lower end of which was driven firmly into the river bed. The recording apparatus was thrown in or out of action by a pull on a string. It was necessary to lift the meter to the surface to read the revolutions, which was a great drawback to its use, as was also the fact that dirt in the water retarded the train of wheels. Lepont sought to remove these difficulties by bringing the recording apparatus to the surface, away from the dirty water and where it could be read without lifting the meter. He did this by introducing a vertical shaft or rod with beveled gearing, but this increased the friction so that it could only be used in shallow water. Baumgarten, Saxton, and others following modified its form somewhat, but did not materially improve it. Brewster made one in which the axis of the meter was a long screw and the revolutions were given by the distance the wheel traveled along this screw. The friction was thus lessened, but the meter had to be stopped before the wheel traveled the whole length of the screw. It was a long step in advance that was taken by D. F. Henry when he applied an electric recording device to the meter, for by it the difficulties that Lepont and others sought to avoid are very successfully overcome, the friction of the train of recording wheels being entirely done away with. If a recording device is used it is worked by a spring and not by the meter, and, furthermore, it is not necessary to lift the meter to the surface to read the number of revolutions, which are recorded electrically, or may be counted from the indications of a buzzer. The electric meter as used by Henry is shown in fig. 10, page 35; that used by Moore in fig. 3.

PRICE ELECTRIC CURRENT METERS.

Meters in use at the present day may be divided into two classes: (1) Those in which the revolving part turns about a horizontal axis, and (2) those in which it revolves about a vertical axis. The former class is illustrated by the Haskell and Fteley meters, shown in figs. 4 and 5, respectively, and the latter class by the Price meter, shown in Pl. I. Meters of the latter class have some advantages over those of

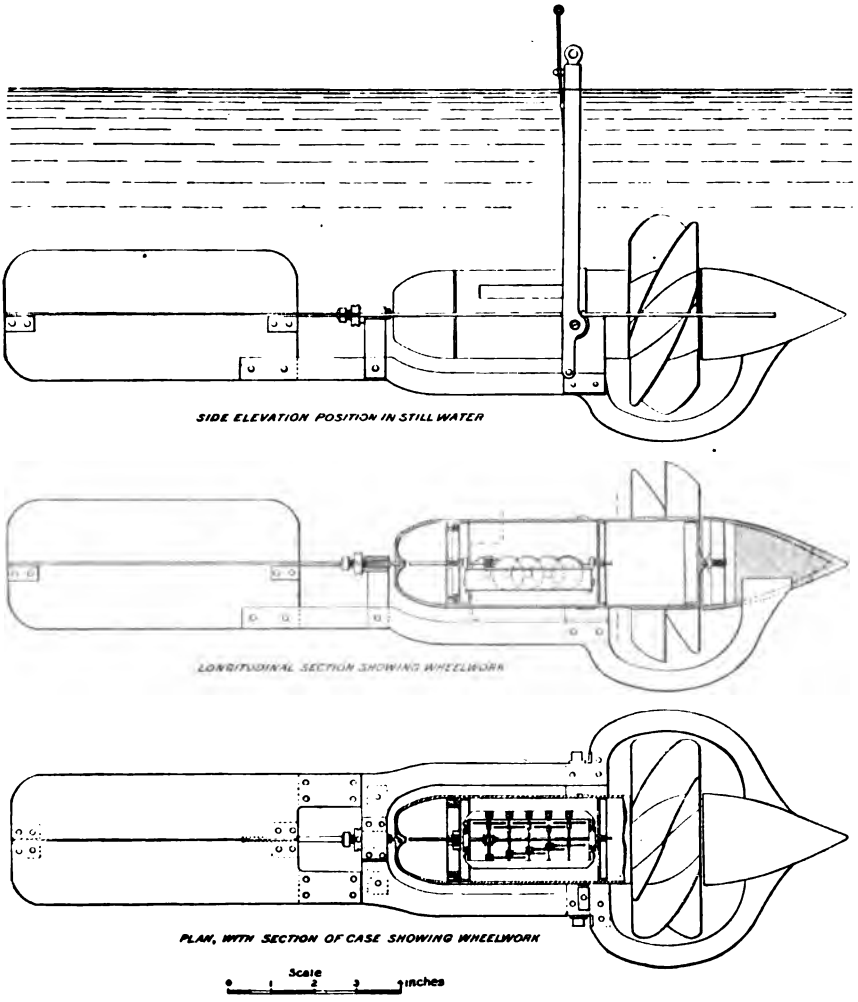

FIG. 3.—Moore current meter.

the former class: First, friction is usually less, since it nearly all comes on one point, and this point can be protected from the action of grit in the water and from jars; and, second, for a given high velocity the wheel will not revolve as rapidly as the wheel of a meter of the first class under the same conditions, and at the same time the wheel will start in a less velocity than will the wheel of a meter of

the former class, so that both low and high velocities can be measured more accurately with meters of the second class than with those of the first class. For example, by comparing the rating table of Has-

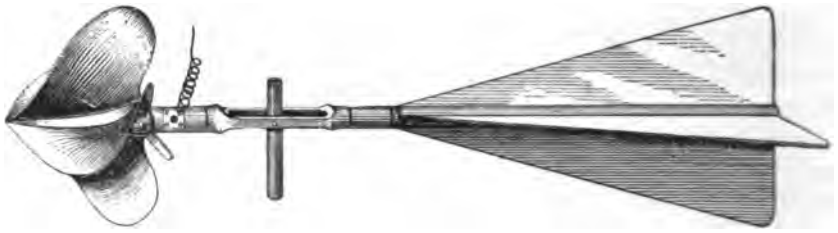

FIG. 4.—Haskell current meter.

kell meter No. 3 on page 81 with that of small Price meter No. 363 on page 42, we find that the former requires a velocity of about 0.20 foot per second to start it, and that it makes 6.5 revolutions per second in water having a velocity of 7 feet per second, while the latter requires a velocity of about 0.08 foot per second to start it and makes only 3.02 revolutions in water having a velocity of 7 feet per second. A rate of 3 revolutions per second can easily be recorded or counted, but a rate of 6.5 revolutions per second can not be counted or easily recorded.

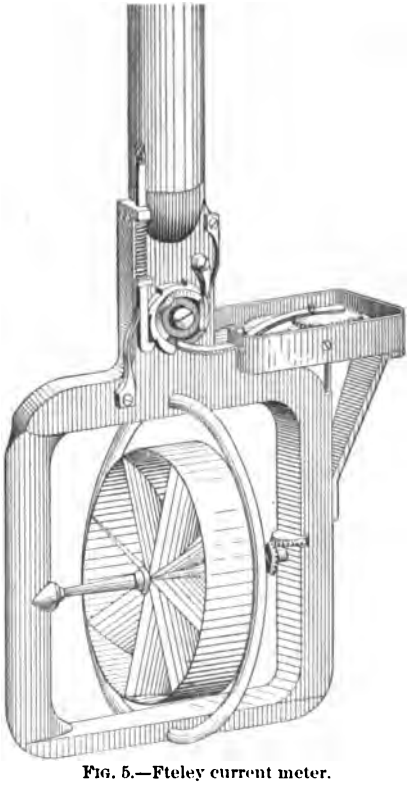

FIG. 5.—Fteley current meter.

The disadvantages of the current meter are, briefly, as follows: (1) It can not be used where there is much floating grass or weeds, as on canals where the weeds are cut and allowed to float in the water, float rods being much better in such cases; (2) it requires rating before use and frequently thereafter; and (3) it must be used with care. These last two conditions, however, are true for all instruments used for measuring velocity indirectly.

The meter that has no electrical device for indicating to the observer the revolutions in a given time, and that must be lifted out of the water to read the revolutions, is a thing of the past. Too much time is lost in making the readings. The acoustic meter is not a success; the clicks are not sufficiently loud to be heard when the water is deep and the place noisy.

The advantages of the meter are as follows: (1) It gives integrated or mean velocity (any kind of a float gives the velocity of a comparatively few particles at some stage of a pulsation of the water, and not mean velocity); and (2) it can be used on streams of all sizes.

For large streams the choice of instruments is between the current meter and the double float, and the meter is preferable except where a large amount of weeds and grass is carried along in the water. For artificial channels the choice is between the meter and the float rod, and the meter has the preference except where there is a considerable amount of floating weeds, and possibly for very slow velocities also, but the latter is still in doubt. For velocities of 0.35 foot per second the meter will give the mean velocity as accurately and more rapidly than the rods, and at less cost, for at least four persons are necessary with the rod method, whereas one person, or at most two persons, can make the gaging with the meter, and the expense of moving the equipment from place to place is much greater for the rods than for the meter.

SLOPE FORMULA.

The velocity of a stream can be computed from measurements of the slope of the surface, the dimensions of the cross section, and a knowledge of the roughness of the bottom and sides. The formula which is now almost universally used for this purpose was devised by Kutter in 1869[a] and is as follows:

$$V=\left[\frac{41.6+\frac{1.811}{n}+\frac{.00281}{s}}{1+(41.6+\frac{.00281}{s})\frac{n}{\sqrt{Rs}}}\right]\sqrt{Rs}$$

In this formula V=velocity, in feet per second; s=surface slope=$\frac{h}{l}$, h being the fall of the surface in the distance l; R=hydraulic radius=$\frac{F}{w}$, F being the cross-sectional area and w the wetted perimeter; and n a ratio whose value varies from 0.009 for a well-planed wooden channel to 0.035 for a channel overgrown with weeds or covered with stones.

This formula is said to apply to streams of all sizes, from creeks and sewers to large rivers like the Mississippi, its constants being determined from Bazin's gagings of small channels, from Humphreys and Abbot's gagings of Mississippi River, from Cunningham's gagings of the Ganges Canal, and from gagings by many others.

The disadvantage of the method is that it is very difficult to measure accurately the surface slope (s) of large rivers. T. G. Ellis says[b] that on Connecticut River the slope of parts 100 to 400 feet long was

[a] Ganguillet, E., and Kutter, W. R., Flow of Water in Rivers and Other Channels.
[b] Trans. Am. Soc. Civ. Eng., vol. 11, p. 23.

very carefully measured and the discharge computed, using the best modern slope formulæ. The discharge found from these formulæ differed by 50 to 250 per cent from that found by gaging. He thinks the slope is so uncertain an element that slope formulæ are of little value. Captain Cunningham, after an examination of five hundred slope measurements of the Ganges Canal, says:[a] "It [the slope of surface measurement] is so delicate a matter that the results are of doubtful use." He found that the slope was very different at different parts of a reach from 1 to 2 miles long, and that the slope at opposite sides differed 50 per cent. Ellet says,[b] in regard to the slope of the Mississippi River: "It not unfrequently happens that while the mass of the water which its channel bears is sweeping to the *south* at a speed of four or five miles per hour, the water near the shore is running *to the north* at a speed of one or two miles per hour. It is no unusual thing to find a swift current and a corresponding fall on one shore toward the south, and on the opposite shore a visible current and an appreciable slope toward the north."

Only in very rare cases is it advisable to use this method.

MEASUREMENT OF SURFACE FLUCTUATIONS.

Any change in the elevation of the surface of a stream affects the discharge in two ways: The cross-sectional area is changed and as a rule the velocity also is changed.[c] It is necessary, therefore, in accurate work to measure the surface fluctuations with great care. Wave action, due to wind or other causes, should be eliminated.

There are two classes of instruments in use at the present time for indicating surface elevations: (1) Self-registering gages, which give a continuous record of the changes in surface elevation; and (2) a fixed scale from which the elevation of the surface is read at any time, or a moving scale with a fixed index from which the elevation of the surface is read.

The United States Lake Survey self-registering water gage, shown in Fig. 6, is a good illustration of instruments of the first class. The center pin is connected to a float on the surface of the lake, and its motion is proportional to the change in elevation of the float. This motion is traced by the pen on a sheet of paper on the drum, which is moved at a uniform rate by clockwork. The two outer pens record the time. Fig. 7 is a reproduction of the record made by this instrument on May 17, 1899. The elevation of the surface at any instant is given by the corresponding ordinate of this curve. This figure illus-

[a] Proc. Inst. Civ. Eng., Vol. 71, p. 11.

[b] Humphreys and Abbot, Report on the Mississippi River, p. 218.

[c] On some streams there are places where the mean velocity remains nearly constant for several feet variation in stage of river. This is true of the Verdigris River at Liberty, Kans. See Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1899, p. 374.

trates the need of an accurate record of the change in stage, for it shows a change of surface elevation of 2.5 feet in one and five-tenths hours.

The hook gage, shown in fig. 8, is a good illustration of a gage of the second class. The frame is fastened firmly to a post in the water,

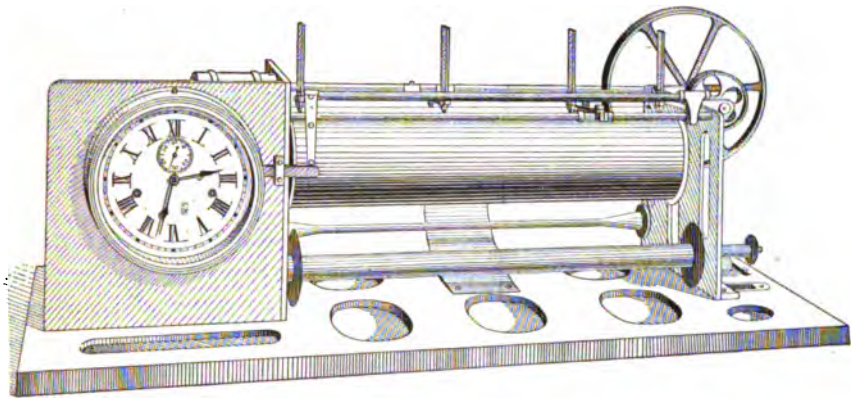

FIG. 6.—Self-registering water gage used by United States Lake Survey.

and the elevation of the zero reading is determined. By turning the milled nut the hook is raised until its point comes to the surface of the water. This can be done very accurately in still water, by means of the little spot of light on the point of the hook. The scale is then read, and from it the surface elevation is found.

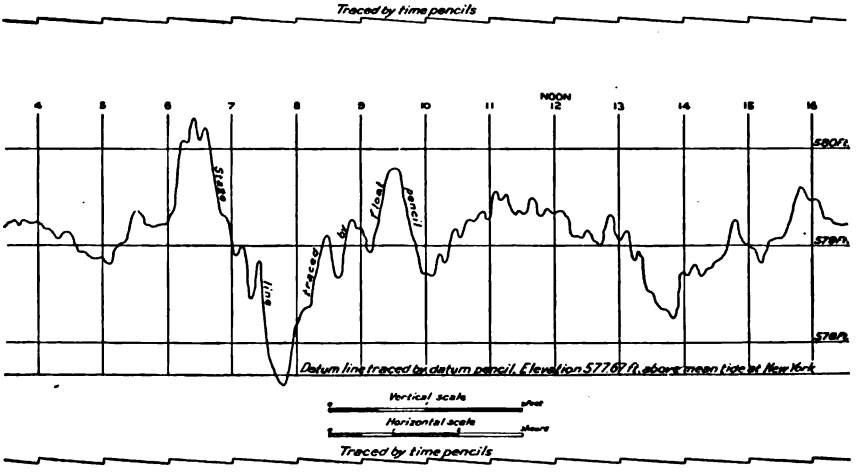

FIG. 7.—Record made by self-registering water gage used by United States Lake Survey.

The water gage used by the United States Board of Engineers on Deep Waterways in 1897–98, on Niagara River at Buffalo, N. Y., was of the box-with-float type. The box was of wood, 7 inches high, 7

inches wide, and 7 feet long, with a closed bottom and a removable cover. The box was fastened firmly in the water, with half its length below the surface. Water was admitted through one or more of three ¼-inch holes, allowing the float to change its elevation as the river stage changed and at the same time eliminating the effect of waves. The float was a 2-quart bottle with a 7-foot staff, marked to feet, tenths, and hundredths, wedged into it. This staff extended up through the cover, and was marked so as to give the rise of the water surface above a certain plane as read at the cover of the box.

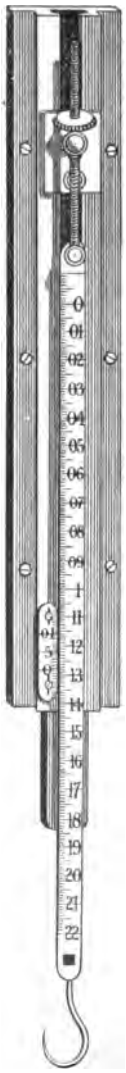

FIG. 8.—Hook gage for measuring surface fluctuations.

VERTICAL VELOCITY CURVES.

The relation between velocity and depth in a vertical section of a stream parallel to the thread of the current, or the vertical velocity curve is very desirable. From such a curve the ratio of velocity at any depth to mean velocity can easily be found and the discharge be computed from observations at one depth only in verticals.

Much effort has been expended in investigating this relation and the change in it for change in river stage. As might be expected, the results of experiments in this direction do not agree. Each investigator adopts a new form of curve to fit his observations. In 1791 Waltman made some observations on the Rhine, from which he concluded that the vertical velocity curve is a reversed parabola with its vertex below the bed of the river. In 1820 De Fontaine found the greatest velocity of the Rhine to be at the surface. He states that two inclined right lines intersecting about mid depth satisfy his vertical-curve observations. In 1824–1826 Racourt made observations on Neva River, from which he concluded that this curve is an ellipse whose minor axis is a little below the surface. Funk adopted a logarithmic curve for his observations on the Weser in 1820. In 1844 Boileau found this curve for a small canal to be a parabola with its axis near the surface. Darcy and Bazin found it to be a reversed parabola for their experiment canal, the perimeter changing with the character of the bed. For Mississippi River, Humphreys and Abbot found it to be a parabola whose axis is three-tenths depth below the surface. Henry found it to be an ellipse for St. Clair River, while Baumgarten says that no simple curve will fit the observations in a vertical. From the results of these investigations and others that might be

mentioned it appears that the shape of the vertical velocity curve depends much on local conditions, such as roughness of bed, slope, ratio of depth to width, wind, obstructions, etc. Two facts must be borne in mind, however: (1) All the vertical-velocity-curve observations obtained with a single meter will not when plotted fall on any one line, but the results will be scattered more or less, and it will be possible to pass a number of curves among them, one satisfying the observations about as well as another; and (2) there is some part of each of the curves mentioned which if placed by the side of some part of another curve will coincide with it closely; so that it is quite possible for any or all of these curves to fit more or less closely a set of imperfect vertical-velocity-curve observations.

There are two methods of obtaining vertical velocity curves with current meters: (1) The single-meter method, and (2) the multiple-meter method. In the former a meter is held at as many points in a vertical as desired—at each tenth of the depth, for example—for a time sufficiently long to eliminate the effect of pulsations. This requires one or more hours to a vertical, during which time a permanent change may occur in the mean velocity. In the latter method, however, this difficulty is overcome, for enough meters are used simultaneously to give the whole curve in one operation.

In the vertical-velocity-curve work on St. Clair River, under the direction of E. E. Haskell, United States assistant engineer, eleven meters were used at a time. These were fastened between two cables, which were attached to a 200-pound sinker at the lower end and to a drum at the upper end, and operated from a catamaran. The lower meter was 1.75 feet above the bottom; the other ten were placed at each tenth of the depth. Each meter was connected with an electric register, and by means of a switch all of the circuits could be opened in one operation. The observer, with a stop watch in one hand and the switch handle in the other, started the watch and at a given instant closed all of the circuits through the meters and registers. At the end of the observation he stopped the watch and opened the circuits at the same instant. One observation of six hundred seconds was made with the meters arranged as described, and from one to ten others of one hundred seconds each with the meters arranged in other ways in each vertical.

There should be some criterion for the rejection of doubtful observations and the grouping of vertical velocity curves. The following table gives the position of the vertex of sixty-nine vertical velocity curves obtained on Mississippi River by Humphreys and Abbot, and illustrates this need.

Table showing position of vertex of vertical velocity curves obtained on Mississippi River.

Depth.	First series.		Second series.		Total.	
	Number of verticals.	Number of floats.	Number of verticals.	Number of floats.	Number of verticals.	Number of floats.
Surface	12	63	8	17	20	[illegible]
One-tenth	11	46	1	2	12	[illegible]
Two-tenths	5	33	.....	.....	5	[illegible]
Three-tenths	3	20	1	3	5	[illegible]
Four-tenths	.....	.....	4	9	4	[illegible]
Five-tenths	2	22	1	3	3	[illegible]
Six-tenths	5	30	1	3	6	[illegible]
Seven-tenths	.....	.....	6	18	6	[illegible]
Eight-tenths	.....	.....	6	15	6	[illegible]
Nine-tenths	.....	.....	2	5	2	[illegible]
Bottom	1	8	.....	.....	1	[illegible]
Sum	39	222	30	75	69	[illegible]

The vertex of 34 per cent of the curves in the foregoing table is at or below mid depth of the river. Using all of them the vertex is three-tenths depth below the surface. This combination of curves so dissimilar is, to say the least, very questionable. Those in which the vertex is at or below seven-tenths depth should be rejected, or at least not combined with the others, for they are due either to errors or to abnormal conditions, and should not be used in deriving a general law.

Vertical velocity curves obtained in different parts of the discharge section for different depths are frequently combined by combining the velocities at each tenth of the depth. This assumes that the curves are similar in all parts of the discharge section, which is not true except in rare cases.

There is very little available data on the change in the vertical velocity curve with change of river stage. In the data on this point that have come to the writer's observation the change in stage is small and the inference to be drawn not conclusive. The effect of roughness of bed decreases, while that of slope and hydraulic radius increases with increase of stage. The effect of these on the shape of the curve can only be determined by careful measurements in which there is a wide range of stage.

PULSATION OF MOVING WATER.

Attention has already been called to the fact that one of the reasons for the difficulty in measuring accurately the velocity of water is that the velocity at any given point in a stream is constantly changing. This is true of both natural and artificial channels of all sizes. Captain Cunningham, who observed this pulsation of moving water in his experiments on the Ganges Canal, of India, states that he considers fifty repetitions with float rods necessary in order to get a good average value of the velocity in one vertical of a canal.[a] J. B. Francis has made numerous measurements of the velocity with float rods in a rectangular wooden flume at Lowell, Mass., every precaution being

[a] Proc. Inst. Civ. Eng., vol. 71, p. 8.

taken to have the usual causes of irregularity of motion suppressed. The time of run was obtained by the use of an electric telegraph and a chronograph, and the depth was very carefully measured with a hook gage, and yet he found that the velocity shown by these floats varied from 8.57 per cent above the mean to 11.4 per cent below it. He says: "Similar and probably greater variations occur in different parts of the depth from the same cause."[a] Adam Baum has made some observations on the variation of velocity in the Rhine near the Bridge of Constance.[b] He fixed a current meter in the river and noted the time of each 100 revolutions of the meter wheel for a period of two hours. The results show a continual change in the velocity for a constant depth of water. He concludes that for an accurate measurement of velocity it is necessary to extend the observations over a period of one hour at each point.

Professor Unwin has made some observations with a current meter on the variation in velocity in the river Thames.[c] He noted the time of each 100 revolutions, and upon plotting the time and the revolutions found that they gave a very irregular curve. The mean of each 500 revolutions, however, when plotted with the corresponding time gave a rather regular curve. The following are some of the results of Professor Unwin's observations:

Variation in velocity at three points of a vertical of Thames River as found by Unwin.

At 0.5 meter depth; velocity found from—		At 3 meters depth; velocity found from 100 revolutions.	At 6 meters depth; velocity found from 100 revolutions.	Variation $\left(\frac{V_m - V}{V_m}\right)$.[a]		
100 revolutions.	500 revolutions.			At 0.5 meter depth.	At 3 meters depth.	At 6 meters depth.
Met. per sec.	*Met. per sec.*	*Met. per sec.*	*Met. per sec.*	*Per cent.*	*Per cent.*	*Per cent.*
1.909	1.877	1.861	1.111	−1.8	+6.6	− 3.6
1.942	1.846	2.030	1.069	−3.6	- 1.9	+ 0.3
1.987	1.846	1.942	0.977	−6.0	+2.6	+ 8.9
1.942	1.846	2.030	1.180	−3.6	−1.9	−10.1
1.861	1.832	2.030	1.044	+0.7	- 1.9	+ 2.6
1.861	1.852	2.060	0.937	+0.7	--3.4	+12.6
1.887	1.823	1.909	1.021	−0.7	+4.2	+ 4.8
1.861	1.787	1.909	0.998	+0.7	+4.2	+ 7.0
1.861	1.809	1.861	0.977	+0.7	+6.7	+ 8.9
1.861	1.787	2.030	1.091	+0.7	1.9	− 2.1
1.942	1.861	2.125	0.900	−3.6	−6.7	+16.1
1.760	1.823	2.030	1.094	+6.1	−1.9	-- 2.1
1.831	.....	2.086	1.021	+2.4	-4.7	+ 4.7
1.719	.....	1.991	1.180	+8.3	0.0	−10.1
1.942	.....	2.086	1.079	−3.6	−1.7	− 0.7
1.831	.....	1.909	1.472	+2.4	+4.2	- 37.4
1.875	.....	1.993	1.072	.....	.....	.....

[a] V=velocity; V_m = mean velocity.

It is seen that during the period required for the meter to make 1,600 revolutions the greatest variations from the mean velocity at 0.5 meter depth are +8.3 per cent and −6 per cent and at 6 meters depth +16.1 per cent and −37.4 per cent. The greatest difference in

[a] Trans. Am. Soc. Civ. Eng., vol. 7, p. 117.
[b] Proc. Inst. Civ. Eng., vol. 71, pp. 456–459.
[c] Op. cit., pp. 348, 349.

velocity, as shown by two consecutive 100-revolution periods, is 12 per cent at 0.5 meter depth and 36.7 per cent at 6 meters depth. A velocity measurement of 100 revolutions 0.5 meter below the surface may be in error 8 per cent and one 6 meters below may be in error 37 per cent.

D. F. Henry[a] has observed this fluctuation in velocity at a given point in large and in small streams. He says: "The lesser fluctuations have a duration of 30 to 60 seconds and the larger ones from 5 to 10 minutes. They do not seem to be synchronous with the surface fluctuations, and are smaller at the surface than at the bottom."

Harlacher[b] found the velocity near the surface of the Rhine to vary 20 per cent in a few seconds, and near the bottom he found it to vary 50 per cent in the same period.

Marr's simultaneous observations of velocity in the Mississippi River at Burlington, Iowa,[c] with five current meters are described on pages 49, 50. The following table gives the velocity, as shown by two of his current meters, one near the surface and the other near the bottom, for consecutive periods of one minute each. The depth in the first section is 11 feet, in the second section 27.6 feet, the meter being 4 feet above the bottom:

Variation in velocity per minute in Mississippi River at Burlington, Iowa.

Simultaneous observations in section No. 1.				Simultaneous observations in section No. 2.			
Velocity 1 foot below surface (V).	Variation $\left(\frac{V_m - V}{V_m}\right)$.	Velocity 9.1 feet below surface (V_m).	Variation $\left(\frac{V_m - V}{V_m}\right)$.	Velocity 3.6 feet below surface (V).	Variation, $\left(\frac{V_m - V}{V_m}\right)$.	Velocity 23.6 feet below surface (V_m).	Variation $\left(\frac{V_m - V}{V_m}\right)$.
Ft. per sec.	*Per cent.*	*Ft. per sec.*	*Per cent.*	*Ft. per sec.*	*Per cent.*	*Ft. per sec.*	*Per cent.*
2.242	+2.0	1.724	0.0	2.286	+4.8	1.966	+ 2.4
2.267	+0.9	1.675	+ 2.9	2.353	+1.3	1.817	- 9.8
2.252	+1.1	1.761	— 2.2	2.437	−2.3	1.985	+ 1.5
2.236	+2.2	1.744	— 1.2	2.300	+3.5	2.008	- 0.3
2.267	+0.9	1.797	— 4.3	2.404	- 0.9	2.065	— 2.5
2.299	- 0.5	1.766	- 2.4	2.378	+0.2	2.119	— 5.2
2.289	0.0	1.772	— 2.8	2.414	−1.3	1.926	+ 4.4
2.274	+0.6	1.819	— 5.5	2.392	- 0.4	2.008	+ 0.3
2.211	+3.3	1.712	+ 0.7	2.313	+3.0	1.983	+ 1.5
2.224	+2.8	1.644	+ 4.7	2.296	+3.8	2.081	— 3.3
2.280	+0.3	1.698	+ 1.5	2.383	0.0	2.104	- 4.5
2.239	+2.1	1.915	--11.1	2.397	−0.6	2.276	−13.3
2.280	+0.3	1.670	+ 3.1	2.420	−1.6	2.092	— 3.9
2.289	0.0	1.775	-- 3.1	2.361	+0.9	2.008	+ 0.3
2.321	−1.5	1.582	+ 8.3	2.278	+4.4	2.047	— 1.6
2.420	−6.0	1.664	+ 3.5	2.445	−2.6	1.848	+ 8.3
2.445	--6.9	1.741	— 1.0	2.459	−3.2	2.024	— 1.0
2.308	−0.9	1.717	+ 0.5	2.428	−1.9	2.070	— 2.8
2.333	−2.0	1.800	— 4.4	2.456	−3.1	1.796	+10.8
2.264	+1.0	1.695	+ 1.7	2.460	−3.2	2.055	— 2.4
2.327	−1.3	1.812	— 5.1	.....	.....	.....	.....
2.311	−1.0	1.689	+ 2.0	.....	.....	.....	.....
2.305	−0.8	1.792	— 4.0	.....	.....	.....	.....
2.311	- 1.0	1.627	+ 5.7	.....	.....	.....	.....
2.277	+0.4	1.554	+ 9.9	.....	.....	.....	.....
2.274	+0.6	1.678	+ 2.7	.....	.....	.....	.....
2.292	−0.2	1.731	— 0.4	.....	.....	.....	.....
2.271	+0.7	1.704	+ 1.1	.....	.....	.....	.....
2.321	−1.5	1.698	+ 1.5	.....	.....	.....	.....
2.261	+1.1	1.723	0.0	.....	.....	.....	.....
2.217	+3.6	1.757	-- 1.9	.....	.....	.....	.....
2.280	+0.3	1.743	— 1.1	.....	.....	.....	.....
2.287	.....	1.724	.....	2.383	.....	2.014	.....

[a] Jour. Franklin Inst., vol. 62, p. 323.

[b] Trans. Am. Soc. Civ. Eng., vol. 12, p. 311.

[c] McKenzie, A., Report on Current Meter Observations, Burlington, Iowa, 1884.

It is seen that in the first section the greatest departures of velocity from the mean are +3.6 per cent and −6.9 per cent at 1 foot below the surface, and +9.9 per cent and −11.1 per cent at 9.1 feet below the surface, or 2 feet above the bottom. The greatest difference in velocity obtained from two consecutive 1-minute periods is 6 per cent at 1 foot below the surface and 12.5 per cent at 9.1 feet below the surface, or 2 feet above the bottom. In the second section the greatest departures of velocities from the mean are +4.8 per cent and −3.2 per cent at 3.6 feet below the surface, and +10.8 per cent and −13.3 per cent at 23.6 feet below the surface, or 4 feet above the bottom. The greatest difference in velocity obtained from two consecutive 1-minute periods is 7 per cent at 3.6 feet below the surface and 13.6 per cent at 4 feet above the bottom. In the first section a velocity measurement near the surface may be in error 7 per cent and a measurement 2 feet above the bottom may be in error 11 per cent. In the second section a velocity measurement 3.6 feet below the surface may be in error 4.8 per cent and a measurement 4 feet above the bottom in error 13.3 per cent.

In 1899, while measuring the discharge of the St. Clair River, Mr. L. C. Sabin, assistant engineer, made some experiments to determine the pulsations or fluctuations in the velocity of the river at the discharge station. Four meters were used, placed 50 feet apart and at the same depth. In the first series of observations the meters were in a line across the river and at right angles to the current; in the second series they were in a line with the axis of the current. Simultaneous readings of the four meters were taken every fifteen seconds for several periods of ten minutes each. The results were plotted, using time as abscissæ and velocity as ordinates, and a curve drawn for each meter, showing the fluctuations in velocity in each fifteen seconds of time at four points 50 feet apart. These curves have two sets of waves—small ones of fifteen to sixty seconds amplitude and larger ones of three to six or more minutes amplitude. The range of velocity as found from the large waves is in some cases 35 per cent of the mean velocity shown by the meter for ten minutes. The curves for two adjacent meters are at times nearly parallel, but they soon diverge and cross each other occasionally. The curves for the meters in a line with the axis of the river resemble one another more closely and are more nearly parallel than those of the meters in a line across the river. These experiments indicate that the pulsations are very limited in extent in a direction at right angles to the current, but that they can be traced for some distance in the direction of the thread of the current. The whole depth of the river is affected by them, and their effect decreases from the bottom toward the surface.

The practical lesson to be drawn from the experiments by Sabin and others is that velocity observations of short duration are of little

value unless they are numerous and well distributed over the discharge area. In order to eliminate the effect of the pulsations, each observation should extend over a period of from six to ten minutes. Long single observations and rapid fluctuations of water-surface elevation require that discharge be found from single observations in comparatively few verticals, and that the ratio of velocity at this depth to mean velocity be known from vertical velocity curves. This emphasizes the need of a thorough investigation of the relation between velocity and depth in a vertical-longitudinal plane and the change in this relation with change of river stage.

METHODS OF MEASURING AND COMPUTING STREAM FLOW.

WEIRS.

The discharge of a stream can be obtained by causing it to flow over a weir or dam, the coefficient of which is known, by the substitution of observed data in a weir formula. Three sharp-crested or standard weir formulæ are in use, viz, Francis's, Fteley and Stearns's, and Bazin's. They differ much in form, and each is derived from experiments on small weirs with low heads. A brief discussion of each formula is here given because of its bearing on the accuracy of discharge measurements. Bazin has determined the values of the coefficient of discharge over weirs of many other shapes of crest, and the Cornell University experiments of the Board of Engineers on Deep Waterways have extended the values of the coefficients of a few of these to the higher heads. Time and space, however, will not permit more than the mention of them here.[a]

The weir is the most accurate method of measuring discharge yet used for small streams if the coefficient of the weir be accurately known. The head on the weir and the velocity of approach are the only variables, and these can easily and accurately be measured. The disadvantages are (1) the cost of constructing a weir, or the difficulty of finding one in use whose shape closely resembles one whose coefficient is known, and (2) the limited range of values of the coefficient of a weir of any shape and the limited number of shapes that have been investigated.

FRANCIS'S EXPERIMENTS AND FORMULA.

In 1852 J. B. Francis made numerous very careful measurements of the volume of water passing over small weirs and the corresponding heads at Lowell, Mass.[b] His measuring tank was a canal lock which had a capacity of 12,138 cubic feet for a depth of 9.5 feet; the greatest length of crest was 10 feet; the head on the weir varied from 5 to 19 inches, and was measured with two hook gages 6 feet upstream from

[a] A discussion of these experiments, which were conducted by George W. Rafter, will be found in Trans. Am. Soc. Civ. Eng., vol. 44, pp. 220–398.

[b] Francis, J. B., Lowell Hydraulic Experiments, p. 133.

the crest; the width of the channel was about 14 feet. His formula, derived from these experiments, is as follows:

$$Q=\tfrac{2}{3}\times 0.622\ h_2(b-\tfrac{1}{10}\ nh_2)\sqrt{2\ gh_2}=3.33\ (b-\tfrac{1}{10}\ nh_2)h_2^{\frac{3}{2}}.$$

Q=discharge in cubic feet per second; h=head on weir, or difference in elevation of weir crest and water surface; n is a constant whose value is 0 when both ends of the weir are flush with the sides of the channel, 1 when one side only is flush, and 2 when neither side is flush; g=acceleration of gravity. (See fig. 9.) The head on the weir should be not less than 0.5 foot nor more than 2 feet, the depth of water in the canal should be at least three times the head on the weir, and the air should have free entrance under the sheet. The velocity of approach of the water to the weir must be measured and the head corrected for it. The formula for this purpose is $h_2=[(h+h_1)^{\frac{3}{2}}-h_1^{\frac{3}{2}}]$, h being the measured head, h_1 the head due to the velocity of approach $=\frac{V_1^2}{2g}$ and $V_1=\frac{Q}{b_0h_0}$. (See fig. 9.)

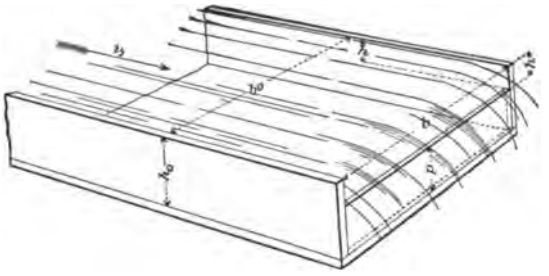

FIG. 9.—Sharp-crested weir.

Hamilton Smith, jr., says of this formula:[a] "It stands in the first rank in reliability."

Francis made eighty-eight experiments with two weirs of 10 and 8 feet length, in addition to those to derive his formula, and found that the greatest percentage difference between his measured discharge and the discharge computed by his formula was 0.9 per cent for the mean of two experiments and 0.68 per cent for the mean of eight experiments; for the other eleven means the difference is less than 0.4 per cent.

FTELEY AND STEARNS'S EXPERIMENTS AND FORMULA.

In 1877 and 1879 Fteley and Stearns[b] measured the discharge over weirs of 5 and 19 feet crest length and the corresponding head on weir, from which they derived the following formula for a standard weir with end contractions suppressed:

$$Q=3.31b\left[h+1.5\frac{V_1^2}{2g}\right]^{\frac{3}{2}}-0.007b.$$

Q=discharge in cubic feet per second, b=length of crest, h=head on weir, V_1=velocity of approach in feet per second. (See fig. 9.)

[a] Smith, Hamilton, Hydraulics, p. 93.

[b] Trans. Am. Soc. Civ. Eng. vol 12, p. 1.

Their measuring tank was a section of the Sudbury conduit having a capacity of 300,000 cubic feet for an increase in depth of 3 feet. The head on the weir was measured with a hook gage 6 feet upstream from the crest. The term $1.5\frac{V_1^2}{2g}$ corrects for velocity of approach. The head on the weir must be not less than 0.06 foot, and the air must have free access under the sheet. This formula is not used as generally as is that of Francis or Bazin. It was used by Fteley and Stearns to compute the discharge of the weir used in their current-meter and weir comparisons. (See p. 58, 59.)

BAZIN'S FORMULA.

In 1886 and 1887 Bazin[a] made numerous measurements of the volume passing over weirs having lengths of from 0.5 meter to 2 meters and the corresponding heads. From these he derived the following formula for a standard weir with end contractions suppressed:

$$Q=\tfrac{2}{3}\mu\left[1+0.55\left(\frac{h}{p+h}\right)^2\right]bh\sqrt{2gh}$$

$$\text{and } \mu=0.6075+\frac{0.0148}{h}\ \text{(in feet)}.$$

This formula is true for any system of units (feet or meters). Q=discharge, b=length of crest, h=head on weir, p=distance of crest above bottom. (See fig. 9.) This formula needs no correction for velocity of approach. The measuring tank was a section of a rectangular channel 200 meters long, 2 meters wide, and 1.2 meters deep. The head on the weir varied from 0.05 meter to 0.6 meter. The air had free access under the sheet. Each of these formulæ will give the discharge to within about 1 per cent if the conditions existing when the observations on which the formula is based are exactly duplicated. Discharges computed from any two of these formulæ may differ 3 per cent.

CURRENT METERS.

If the stream be small, meter measurements are made from a bridge, if one can be found in a suitable location. Bridge piers, however, disturb the natural flow of a stream, as well as distort the vertical and transverse velocity curves and render them of only local application. Discharge measurements made where there are piers in the channel are, as a rule, less accurate than measurements made where there are no piers to obstruct the flow.

In case there is not a bridge at the desired place, the observer wades, if the water be shallow, taking observations of depth and velocity at

[a] Annales des ponts et chaussées, 1888, p. 416.

as many points in the cross section as desired. If the water be deep and swift, a cable is stretched across the stream at a convenient height above the water, and a car or box is suspended from it. From this car, which is moved back and forth across the stream by the observer in it, the depth and velocity of the water are measured. If the current be not swift, a surveyor's chain or a cable can be stretched across the stream temporarily and the depth and velocity be measured from a rowboat at points marked on the chain or cable.

In a large river like the Mississippi, where a steamboat can conveniently anchor, a method that has been used satisfactorily is shown in fig. 10.[a] The boat is anchored in the proper place, and a lead weight of

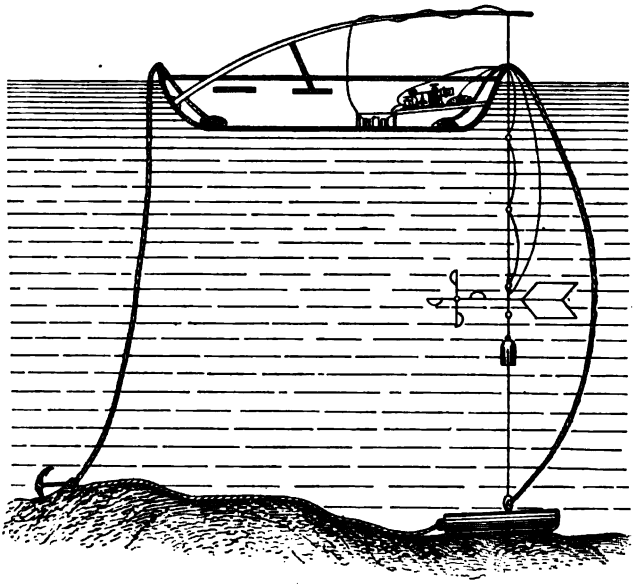

FIG. 10.—Method of using current meter on large river.

50 pounds connected to the anchor is lowered over the stern. A copper wire is connected to this weight and to the spring pole in the boat, and the meter is moved up and down on this wire by a rope. The revolutions of the meter are recorded electrically in the boat, the copper wire being connected to one pole of the battery and an insulated wire connected to the meter and the other pole.

On large rivers where boats can not anchor the measurements are made from a boat towed by a steamboat.[b] The meter is kept in position by two lines of range poles at right angles to each other. The pilot on the steamboat steers it so as to keep it in line with one range, while the engineer, with his hand on the steam valve and his eye on

[a] Jour. Franklin Inst., vol. 62, p. 171. [b] Jour. West. Soc. Eng., vol. 3, No. 3.

the other range, controls the speed of the boat so that it will remain in a fixed position.

In a swift current the meter and its sinker will be carried downstream a distance depending on the weight of the sinker and the depth and velocity of the water. There are two methods of treating this difficulty. One is to apply a special device like that shown in fig. 11

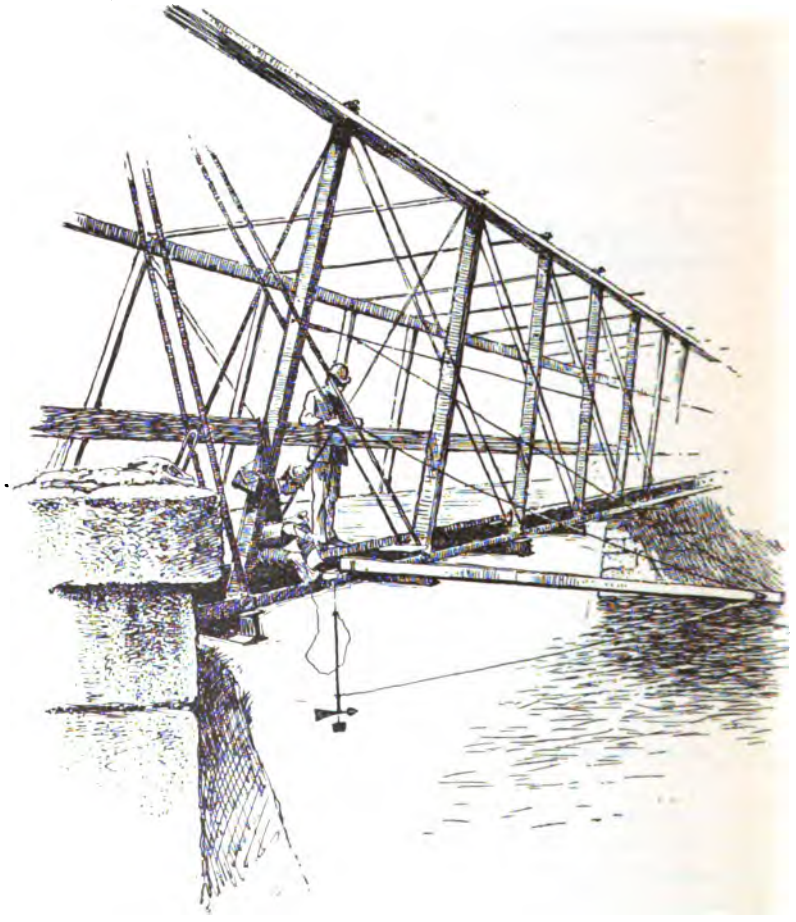

FIG. 11.—Device for holding meter in place.

to keep the meter in place;[a] the other is to compute from the weight of the sinker and the depth and approximate velocity of the water the additional length of cable to use in order that the meter may sink to the required depth.[b] The first is the better for streams of ordinary size. In the device shown in fig. 11 the stay line is attached to the

[a] Twentieth Ann. Rept. U. S. Geol. Survey, pt. 4, 1900, p. 20.
[b] Jour. West. Soc. Eng., vol. 4, No. 6.

meter and is passed over a pulley at the end of the stay pole, its end being held by an assistant. By means of this stay line and the vertical cord the meter can be kept nearly in position. Instead of a stay pole a wire cable is sometimes used to keep the stay line in place.

POINT AND INTEGRATION METHODS.

There are two methods of using a meter in a cross section, as follows: (1) The point method, in which the meter is held at certain points in the cross section for a stated period of time, usually fifty seconds, and the number of revolutions of its wheel during that time are carefully observed, either by counting the clicks of a buzzer or reading a register; and (2) the integration method, in which, instead of holding the meter at certain points, it is moved with a uniform speed through the section, the time and number of revolutions being observed as before. The point method is the more accurate if sufficient observations are taken in the section. The velocity found by the integration method is the resultant of the velocity of the water and that of the meter as it is carried by the observer, and must always be greater than the true velocity. The error increases with the speed of the meter, and also increases as the velocity of the water decreases. When the point method is used the meter may be held at several points in certain verticals, in which case we will call it the ordinary method; or it may be held at the surface, at mid depth, and at the bottom in certain verticals, and the discharge be found from these (see Moore's method, p. 46); or it may be held at the surface and the bottom only in several verticals, and the mean velocity be found from these; or it may be held at three-tenths, at six-tenths, or at mid depth in certain verticals, and the mean velocity be found by applying a factor. The three-tenths-depth point[a] is used because it is said to be the point of maximum velocity in a vertical, and a small error in the position of this point will not affect the result much. The mid-depth point[b] is employed because the factor which is used to obtain the mean velocity is more nearly constant for it than for any other point on a vertical curve. The six-tenths-depth point[c] is used because it is believed to be the depth of the thread of mean velocity.

In the integration method the meter may be moved down several times in certain verticals only, as used by Harlacher,[d] or across the stream at a given depth, as used by Price on the Mississippi River at Carrollton, La.,[e] or diagonally across from one side to the other at the same time that it is moved from the surface to the bottom several

[a] Jour. West. Soc. Eng., vol. 3, No. 3.
[b] McKenzie, A., Report on Current Meter Observations, Burlington, Iowa, 1884.
[c] Proc. Inst. Civ. Eng., vol. 71, p. 60.
[d] Ibid., vol. 67, p. 358.
[e] Jour. West. Soc. Eng., vol. 3, No. 3.

times, as used by the writer and others. Harlacher's apparatus for moving a meter with a uniform velocity is shown in figs. 12 and 13. The meter B is moved up or down on the iron tube AA by the rope D attached to the drum F. E is a bracket holding the pulley e. G is an arm supporting the drum F, and f_3 (fig. 13) is a dial on which is registered the depth. The fan f_4 (fig. 13) and the gearing f_5 regulate the rate of motion of the meter; f_6 is a crank lever for raising the meter, and f_8 a ratchet wheel for arresting the motion. I (fig. 12) is an electric battery, and H (fig. 12) is the registering apparatus.

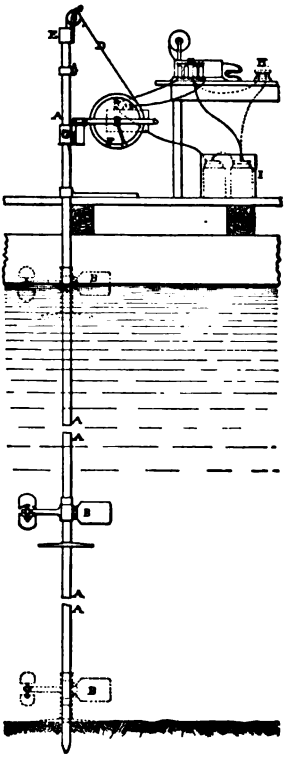

FIG. 12.—Harlacher's apparatus for moving a meter with uniform velocity.

When accurate results are desired the meter should be held with a rod, if the depth be not too great, in which case it is not free to turn sidewise, but keeps its axis parallel to the axis of the stream and can be placed close to the bottom. Ordinarily it is held with an insulated wire, and is free to take the direction of the water at the point where it is held.

COEFFICIENT WORK.

Coefficient work in river gaging consists of obtaining the ratio of the velocity at any point in the depth of the stream (called the index point)—as, for example, the three-tenths-depth point—to the mean velocity in the whole depth. When this ratio or coefficient is known for a vertical, the mean velocity in that vertical can be found by making an observation at the index point and applying the coefficient.

Let C_1, C_2, C_3, etc., be the coefficients for the component parts, F_1, F_2, F_3, etc., of the discharge area F; V_1, V_2, V_3, etc., the observed velocities at the index points of these areas; and V the mean velocity for the whole discharge area. The discharge $Q = FV = C_1F_1V_1 + C_2F_2V_2 + C_3F_3V_3 + \ldots\ldots\ldots\ldots\ldots\ldots (1)$.

A value of each of these coefficients can be found for each foot variation of river stage. The discharge of the stream is then computed by equation (1) from observed values of V_1, V_2, V_3, etc., at the index points, using the set of values of the coefficients which correspond with the observed river stage.

Coefficient work can be done with one current meter, but it can be done more rapidly with two or more meters. When two meters are

used, one is placed at the index point and the other is held for several minutes at as many points in the vertical as desired, and simultaneous readings of the two meters are obtained at each point. The two meters are occasionally held at the same depth and relative readings of the meters are obtained. A chart can be prepared from the ratings of the meters in still water which will give the velocity ratio from the simultaneous readings of the two meters. For example: The index

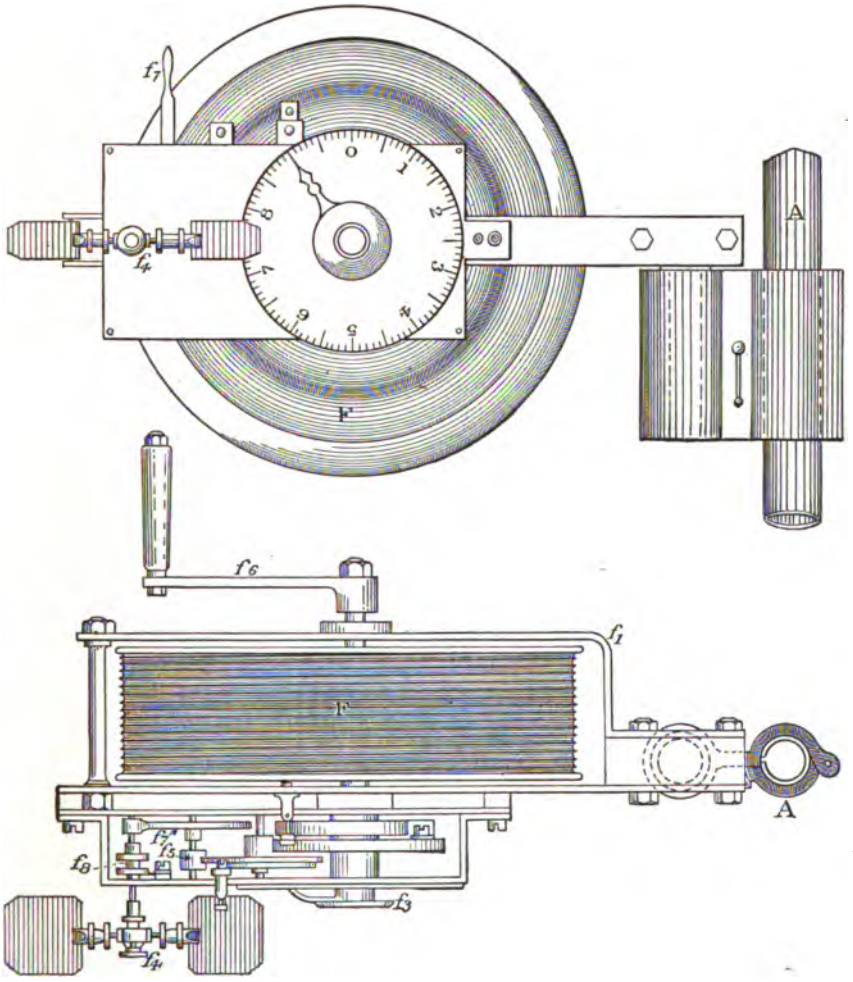

FIG. 13.—Details of Harlacher's apparatus for moving a meter with uniform velocity.

meter at three-tenths depth indicates 2.34 revolutions per second. The other meter, at eight-tenths depth, indicates 2.05 revolutions per second. The chart shows the velocity ratio for these meter speeds to be 0.89. From the velocity ratio at each tenth of the depth the ratio of the mean velocity in the vertical to the index velocity, or C, is found.

On account of the rapid changes of surface elevation of large streams, this coefficient method of gaging is better and more accurate for such streams than the slower one in which velocity observations are made at several points in each vertical.

RATING OF METERS AND CONSTRUCTION OF RATING TABLES.

The accuracy of a discharge measurement depends largely on the accuracy of rating the meter used. Errors of observation are as likely to be too large as too small, and are compensating. Errors in a rating table always have the same sign and are cumulative, hence they should be eliminated or reduced to a minimum.

There are two kinds of rating of a current meter—absolute and relative. From the former the absolute velocity at a single point is found; from the latter the ratio or coefficient of velocity at any two points is found. The former is usually employed in still water; the latter in running water.

Four methods have been used for rating meters: (1) By the use of surface floats; (2) by moving the meter through still water with a known velocity; (3) by observing velocity at many points in a cross section and comparing this with the known mean velocity; and (4) by attaching the instrument to a long arm and revolving it about a vertical axis in a body of still water. The second method is the one now universally used. The meter is suspended from a car or a boat and is moved with a uniform velocity through still water at a depth of 2 or more feet below the surface. The length of a run varies from 100 to 300 feet, with a starting run sufficient to allow the meter wheel to reach the proper speed before entering the run. It is moved across and back over the run to eliminate the effect of velocity in either direction. The rating equipment should be such as to give time of run to the tenth of a second and speed of meter to the tenth of a revolution (a chronograph recording time, revolutions, and the instants of beginning and ending the run should meet this requirement).

The range of velocities employed in rating should be those for which the meter is to be used. If it is to be employed mainly to measure low velocities the speeds in rating should be low. The rating table should not be extended beyond the limits of velocity used in the rating.

The relation between distance passed over by the meter and the corresponding revolutions of the meter wheel is often assumed to be a straight line. It is always a curved line, and must be so on account of friction of bearings and inertia of moving wheel. A velocity of from 0.1 to 0.5 foot per second is necessary to start the wheel. As the velocity increases the effect of friction and inertia becomes proportionately less and the curve approaches a straight line. The relation between velocity and revolutions of wheel, per second, is of the second

degree and concave to the axis of velocity. For velocities of 2 or more feet per second this curve differs little from a straight line.

The results of the rating are usually plotted on squared paper, using revolutions per second as ordinates and velocity as abscissæ, and all observations that fall much from a well-defined line are rejected. If this line be nearly straight between the limits of the observed revolutions its equation is assumed to be of the form $y=a+bx$, y being velocity, x revolutions per second, and a and b constants. If the results indicate a curved line its equation is assumed to be of the form $y=a+bx+cx^2$. There are two methods of preparing the rating table from the observations. The first method is rapid. The most probable curve is drawn among the plotted observations, and the velocities to tenths of a revolution are read from the curve. In the second method the equation of the most probable relation is derived by the method of "least squares" and then the table is prepared from the equation. The writer has found it better to plot values of x and the ratio of y to x (see fig. 29, p. 89) instead of x and y. The reason is that the ratio y to x changes rapidly as x increases and can be read more accurately from this curve than from a nearly straight line. It can be shown mathematically that these curves are approximately branches of hyperbolas.

The second method, which is to derive the equation of the most probable relation between velocity and revolutions by the method of "least squares," first rejecting the observations that fall much outside of a well-defined curve, is the most accurate, but it requires the most time. If the meter is to measure low velocities to within 1 per cent, the second method must be used. It gives n observation equations of the form $y-a-bx=v$ if the relation be linear, or $y-a-bx-cx^2=v$ if it be parabolic. v is a small quantity called a residual, or a residual error. Its value would be zero if the observations were perfect. According to the principle of least squares, the best values of the constants are those that make the sum of the squares of the residuals a minimum.

Squaring each of these n equations and adding, we have

$$\Sigma v^2=v_1^2+v_2^2+v_3^2+\ \ldots\ =[y'-a-bx'-cx'^2]^2+[y''-a-bx''-cx''^2]^2 +[y'''-a-bx'''+cx'''^2]^2+\ \ldots$$

Taking the first derivative of this equation with respect to a, b, and c and placing these equal to 0, we have

$$d\frac{(\Sigma v^2)}{da}=-2\,[y'-a-bx'-cx'^2]-2\,[y''-a-bx''-cx''^2]\ \ldots\ =0$$

$$d\frac{\Sigma v^2}{db}=-2x'\,[y'-a-bx'-cx'^2]-2x''\,[y''-a-bx''-cx''^2]\ \ldots\ =0$$

$$d\frac{\Sigma v^2}{dc}=-2x'^2\,[y'-a-bx'-cx'^2]-2x''^2\,[y''-a-bx''-cx''^2]\ \ldots\ =0$$

From these normal equations the values of a, b, and c can be found.

For the linear function these equations become two in number and have the form

$$a\Sigma x+b\Sigma x^2=\Sigma xy$$
$$na+b\Sigma x=\Sigma y$$

and a and b are found from these equations.

This relation between revolutions per second and velocity, whatever it may be, is not constant. Below are given the ratings of a Price meter, obtained by J. C. Bailey, United States assistant engineer:[a]

Results of four ratings of a Price meter used in 1892 on Niagara River.

Date.	Number of observations.	Velocity.	Meter coefficients.		Remarks.
			a	b	
		Ft. per sec.			
April 28	20	2.5 to 7.3	4.239	+0	Still water; base 150 feet long.
April 29	34	2.1 to 6.7	4.256	+0.061	Do.
May 4	49	1.7 to 7.7	4.046	+0.207	Do.
May 17	43	2.0 to 8.0	4.130	+0.067	Still water; base 300 feet long.
			4.168	+0.0848	

Values computed from two ratings of current meter on April 29 and May 4, 1892.

[April 29, $V_1=4.256R+0.061$; May 4, $V_2=4.046R+0.207$.]

R	V_1	V_2	V_2-V_1	$\frac{V_2-V_1}{V_2}$
0.05	0.274	0.409	0.135	0.314
0.10	0.487	0.612	0.125	0.204
0.15	0.699	0.814	0.115	0.141
0.20	0.912	1.016	0.104	0.102
0.25	1.125	1.219	0.094	0.077
0.30	1.338	1.421	0.083	0.058
0.35	1.551	1.623	0.072	0.044
0.40	1.763	1.825	0.062	0.034
0.45	1.967	2.028	0.052	0.026
0.50	2.189	2.230	0.041	0.018

The foregoing table shows that if on May 4 velocity measurements had been made from 0.05 to 0.50 revolution per second and the rating table of April 29 had been used the results would be in error from 1.8 per cent to 31.4 per cent. It is true that neither set of values of a and b was determined for low velocities. This, however, emphasizes the fact that values of a and b found for high velocities may be greatly in error for the low velocities if the linear relation is assumed. These four ratings of a meter in less than one month show considerable variation in the values of a and b, and indicate that when a meter is in constant use its rating should be examined once a week if accurate work is required.

A rating table that gives velocity to only two places of decimals is not sufficiently accurate for low velocities. To illustrate this we will take the case of the rating table of small Price meter No. 363, rated at Chevy Chase, Md., June 26, 1900. The velocity corresponding with 0.05 revolution per second is given as 0.17 foot per second. This may mean any value from 0.166 foot per second to 0.174 foot per second. This is 2.12 per cent of 0.17. Hence if the computed revolution-

[a] Engineering News, vol. 29, 1893.

were either 0.166 or 0.174 the error introduced by using this rating table would be 2.12 per cent. The following table shows the error for other velocities given in this rating table. If the velocity be 0.25 foot per second it may be in error nearly 2 per cent if found from this table.

Table showing greatest percentage error in rating table of small Price meter No. 363, which gives velocity to only two places of decimals.

Revolutions per second.	Velocity.	Error $\frac{0.004}{V}$.	Revolutions per second.	Velocity.	Error $\frac{0.004}{V}$.
	Ft. per sec.	*Per cent.*		*Ft. per sec.*	*Per cent.*
0	0.06	6.7	0.25	0.63	0.6
0.05	0.17	2.4	0.30	0.75	0.5
0.10	0.29	1.4	0.35	0.86	0.5
0.15	0.40	1.0	0.40	0.98	0.4
0.20	0.52	0.8	0.45	1.09	0.4

Darcy and Bazin employed the first three methods to rate the Pitot tube which they used in their hydraulic investigations on canals.[a] Fig. 2, page 19, shows this instrument as they used it. A discussion of the results of the ratings is given here on account of the light they throw on accuracy tests.

Results of Pitot tube ratings by Darcy and Bazin.

First method.[b]			Second method.[c]			Third method.[d]					
						Designation of experiment.				Channel.	
No. of experiment.	Value of coefficient.	Number of experiments.	Velocity of boat.	Value of coefficient.	Number of experiments.	Series.	No.	Value of coefficient.	Mean velocity.	Shape.	Width.
			Meters.						*Meters.*		*Meters.*
2	0.987	8	0.609	1.040	2	51	1	1.021	0.508	(e)	0.8
3	1.024	8	0.692	1.053	1	51	2	1.010	0.768	(e)	0.8
4	0.981	8	0.785	1.032	3	51	3	1.018	1.028	(e)	0.8
5	1.013	8	0.938	1.033	1	52	1	1.029	0.376	(e)	0.48
6	1.006	8	0.980	1.040	3	52	2	1.000	0.542	(e)	0.48
7	0.988	8	1.120	1.015	1	52	3	1.006	0.694	(e)	0.48
8	1.012	8	1.231	1.028	1	58	4	1.005	1.429	(e)	1.994
9	1.008	4	1.333	1.032	1	59	4	0.969	2.318	(e)	1.994
10	1.008	4	1.385	1.048	1	60	2	0.968	2.571	(e)	1.994
11	1.009	4	1.470	1.029	1	61	4	0.966	1.267	(f)	1.994
12	1.007	4	1.500	1.040	1	62	4	0.965	1.979	(f)	1.994
13	1.015	4	1.611	1.033	2	63	3	0.988	2.199	(f)	1.994
14	1.039	4	1.661	1.027	1	64	1	0.996	0.856	(f)	1.994
15	0.994	4	1.775	1.042	1	64	2	0.988	0.948	(f)	1.994
16	1.007	4	1.819	1.031	3	65	4	0.978	1.511	(f)	1.994
17	1.023	4	1.863	1.027	3	66	1	1.015	1.464	(f)	1.994
.....	.....	.....	1.930	1.039	1	66	2	0.984	1.675	(f)	1.994
.....	.....	.....	1.976	1.025	4	68	4	0.976	1.497	(g)	1.984
.....	.....	.....	2.034	1.037	1	71	6	0.968	1.810	(h)	1.40
.....	.....	.....	.....	.....	.....	72	5	1.017	1.679	(h)	1.40
.....	.....	.....	.....	.....	.....	73	4	0.967	1.612	(h)	1.40
.....	.....	.....	.....	.....	.....	74	4	1.014	1.229	(h)	1.40
.....	.....	.....	.....	.....	.....	84	2	1.014	1.033	(i)	1.990
.....	.....	.....	.....	.....	.....	84	3	0.984	1.246	(i)	1.990
.....	.....	.....	.....	.....	.....	85	1	0.998	1.011	(i)	1.990
.....	.....	.....	.....	.....	.....	85	2	0.997	1.218	(i)	1.990
.....	.....	.....	.....	.....	.....	85	3	0.990	1.473	(i)	1.990
.....	.....	.....	.....	.....	.....	88	1	0.995	0.894	(i)	1.990
.....	.....	.....	.....	.....	.....	88	2	0.986	0.921	(i)	1.990
.....	.....	.....	.....	.....	.....	88	3	0.980	0.955	(i)	1.990
.....	.....	.....	.....	.....	.....	88	4	0.993	1.010	(i)	1.990
	1.006			1.034				0.993			

[a] Darcy and Bazin, Recherches hydrauliques, 1865, pp. 63—70.
[b] By surface floats.
[c] In still water.
[d] In moving water.
[e] Rectangular covered wooden channel.
[f] Rectangular open wooden channel covered with cloth.
[g] Trapezoidal open wooden channel covered with cloth.
[h] Semicircular.
[i] Rectangular.

In the first method the floats were run over a distance of 40 meters and the tube was held with its point 0.02 meter below the surface of the water. C is found from $v=C\sqrt{2g\frac{\Sigma d}{2}}$, v being known from the floats. In the second method the tube was fastened in front of a boat and was drawn through still water at different observed velocities. In the third method the instrument was held at many points in a cross section of an experiment canal 2 meters wide the mean velocity in which was known, and the mean velocity as shown by the tube was computed.

It is seen that the three values of C found by these three methods differ somewhat, although each is the mean of many observations. The value found by the second method (the one now commonly used) is the largest of the three, and is rejected on account of the action of the boat on the instrument. The pull on the boat as it was drawn through the water tipped the point of the tube down slightly, making its indications less than they should be and making C too large. By the first method the mean of experiments Nos. 2 to 8 is 1.0016 and the mean of experiments Nos. 9 to 17 is 1.0122. The difference between these is nearly 1 per cent of the former. The greatest variations from the mean of all the observations by the first method are +1.8 per cent and −2.5 per cent. The value of C found by the third method is 0.993, obtained from thirty-one measurements of discharge of the experiment canal. The tube was held at many points in the cross section. The variations from this mean are +3.6 per cent and −2.8 per cent. Darcy uses the mean of the values found by the first and third methods, which makes $C=1$. The extreme variations from this mean are +3.9 per cent and −3.5 per cent. The velocity of the water during these ratings varied from 0.5 to 2.5 meters per second, or from 1.64 feet to 8.20 feet per second.

FORMULÆ FOR COMPUTING DISCHARGE.

The discharge is a function of the cross section and velocity. Each of these factors must be measured at a sufficiently large number of points in the cross section to give the required degree of accuracy.

For streams from 5 to 10 feet wide the depth should be measured at each foot of width; for streams from 10 to 30 feet wide, at each 2 to 5 feet of width; for widths from 30 to 100 feet, depths should be measured at each 5 to 10 feet, and for widths greater than 100 feet, at each 10 to 25 feet, depending on the width and unevenness of the bed.

The general formula for the discharge per second is

$$Q=F_1V_1+F_2V_2+F_3V_3+\ldots F_nV_n. \qquad (1)$$

F_1, F_2, F_3, etc., are the component parts of the cross section, and V_1, V_2, V_3, etc., the mean velocity in each of these parts. If the

width of these component parts is b and the mean depth in each is d_1, d_2, d_3, this formula can be written

$$Q=b\,(V_1d_1+V_2d_2+V_3d_3+\ldots\ldots). \quad (2)$$

If $d_1=d_2=d_3\ldots\ldots$, that is, if the channel is rectangular in cross section, then formula (2) becomes

$$Q=bd\,(V_1+V_2+V_3+\ldots\ldots). \quad (3)$$

The mean velocity of a component area may be computed from observations at its ends or at its center. Occasionally two consecutive component areas are considered together and the mean velocity for the double strip computed from observations at the ends and center of it. The mean velocity in a vertical is found (1) from a single observation at same depth by applying a coefficient, as a mid-depth velocity observation multiplied by 0.95, or a six-tenths-depth observation multiplied by unity, and (2) from a vertical velocity curve. The latter, as already explained, is found from numerous observations at points in the depth, and the mean velocity in this vertical is then the area inclosed between the vertical curve and a vertical line representing no velocity divided by the depth. Harlacher's method of integrating[a] in a vertical gives each component velocity very readily and quickly; any component velocity is $\frac{RC}{T}$, R being the observed revolutions, T the observed time in seconds, and C the number, which changes from revolutions to velocity.

In measuring the discharge of a large river the component velocities are always found from an observation at some given depth and the use of a coefficient. As the depth or stage of a river is almost constantly changing, it is necessary to make the velocity observation in a short time—one or two hours at most. The value of the coefficient to use to reduce observed velocity at any depth to mean velocity in that vertical should be found from measurement if possible.

The discharge measurements of the Niagara River made in 1897 and 1898 by the United States Board of Engineers on Deep Waterways[b] from the international bridge 3 miles below the head of the river were computed from velocity observations at three-tenths depth, using coefficients whose values were found from vertical velocity curves. The meter stations were about 80 feet apart and were at the middle of each half span of the short spans and at the middle of each one-third span of the long spans. The value of the coefficient for reducing observed velocity to mean velocity ranged from 0.61 to 1.17, being influenced by the piers of the bridge.

The whole cross-section area can readily be computed from the formula:

$$F=\frac{b}{3}[d_1+4\,(d_2+d_4+d_6+\ldots+d_{n-1})+2\,(d_3+d_5+\ldots+d_n)+d_{n+1}]\ldots(4)$$

[a] Proc. Inst. Civ. Eng., vol. 67, p. 358.

[b] Jour. West. Soc. Eng., vol. 4, No. 6.

d_1, d_2, d_3, etc., being depths and b the distance between the points where the depths are measured; n is the number of component areas and should always be an even number. This formula is based on the assumption that the bed is a parabolic arc between each double component area.

When $n=2$, this formula gives the area of two consecutive component areas and becomes:

$$F'=\frac{b}{3}(d_1+4\,d_2+d_3)=\frac{e}{6}(d_1+4\,d_2+d_3) \quad (5)$$

e being equal to $2b$.

A somewhat simpler formula for the area of two consecutive component areas is:

$$F'=\left(\frac{d_1+d_2}{2}+\frac{d_2+d_3}{2}\right)b=\frac{e}{4}\left(d_1+2d_2+d_3\right) \quad (6)$$

T. V. Moore in his discharge measurements of the Thames[a] assumed the vertical velocity curve to be a parabola, and computed its area from the formula $A=\frac{d}{6}[V_s+4V_m+V_b]$, d being the depth, V_s the surface velocity, V_m the velocity at mid depth, and V_b the bottom velocity. The discharge per second flowing between the end sections he computed from the formula $V=\frac{h}{3}[\Sigma A_1+4\Sigma A_2+2\Sigma A_3]$, h being the distance between the consecutive sections, ΣA_1 the sum of the two end sections, ΣA_2 the sum of the odd sections, and ΣA_3 the areas of the even sections. The discharge is found by adding to this the small volume flowing between each end section and the shore.

T. G. Ellis, in computing the discharge of the Connecticut River in 1874, multiplied the velocity at mid depth in each component area by 0.95 to find the mean velocity in that area.[b] In computing the discharge of the Mississippi River in 1882 the velocity at mid depth in each component area was multiplied by 0.91 for the observations taken when the river was covered with ice, and by 0.96 when free from ice, to reduce the mean velocity in a vertical.[c] From 1885 to 1887 the velocity at six-tenths depth was observed in the component areas, and this was taken as the mean velocity in each area.

Harlacher's method of computing flood discharge where the surface velocity only could be measured is illustrated in fig. 14.[d] The surface velocity was measured at a sufficient number of points (1, 2, 3, etc.) in the cross section ACB to determine the surface velocity curve ADB. The line ACB shows the depth at each point and is plotted from soundings taken immediately before or after the flood.

The openings between EF and GH are bridge piers, the measure-

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[a] Proc. Inst. Civ. Eng., vol. 45, p. 220.
[b] Trans. Am. Soc. Civ. Eng., vol. 4, p. 303.
[c] Jour. West. Soc. Eng., vol. 3, No. 3.
[d] Proc. Inst. Civ. Eng., vol. 91, p. 399.

ments being made from a bridge. The product of the surface velocity v_s and the corresponding depth t is found for a sufficient number of points, depending on the irregularity of the bottom, and the curve

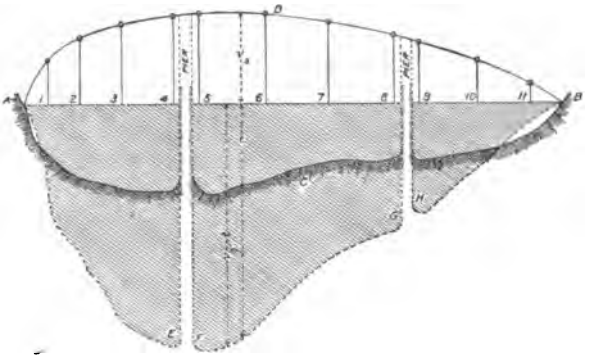

FIG. 14.—Diagram illustrating Harlacher's method of computing flood discharge.

AEFGIIB drawn. The discharge per second is the area between the line *AB* and the curve *AEFGIIB*, less the piers, multiplied by 0.85.

COMPARISONS OF RESULTS WITH DIFFERENT INSTRUMENTS AND METHODS.

SURFACE FLOATS AND PITOT TUBE.

On page 43 we have given the results of three ratings of a Pitot tube by Darcy and Bazin.[a] In the first method the surface velocity was found from surface floats and a Pitot tube, as already described. Ninety-two experiments were made. The results are combined, according to velocity, into seven groups of eight experiments each and nine groups of four experiments each. The value of the coefficient as found from the first seven groups differs from that found from the mean of the last groups by about 1 per cent. The extreme variations of these individual means from the mean of all the experiments are − 1.8 per cent and + 2.5 per cent. The individual values of the coefficient are not given. Their extreme variation from the mean of all must be at least twice that of the individual means from the general mean.

PITOT TUBE AND RECTANGULAR ORIFICES.

On page 43 are given the results of a third rating of a Pitot tube by Darcy and Bazin (third method). This is also a comparison of the discharge of a canal 2 meters wide, as found from rectangular orifices and as found from numerous velocity measurements in a cross section of the canal with the same Pitot tube. The discharge of the rectangular orifices admitting water to the canal was determined by

[a] Darcy and Bazin, Recherches hydrauliques, 1865, pp. 63-70.

experiment for different heads on them. The number of velocity measurements in each experiment or discharge measurement varied with the depth of the water in the canal, being from twenty-seven in the smallest channel, which was 0.30 meter by 0.48 meter, to ninety one in the canal 2 meters wide. The points of measurement were nearest together where the velocity changed most rapidly. The results of thirty-one discharge measurements are given, the mean velocity varying from 0.38 meter to 2.57 meters per second. The extreme variations from the mean of all the values of the coefficient are + 3.6 per cent and — 2.8 per cent.

DOUBLE FLOATS AND CURRENT METERS.

ELLIS'S EXPERIMENTS.

In 1874 T. G. Ellis made numerous velocity measurements on the Connecticut River at Thompsonville, Conn., with current meters and with double floats like that shown in fig. 1, p. 15.[a] Vertical velocity curves were obtained in planes parallel with the axis of the river and 100 feet apart, with the meters and with the floats. Each of these curves was plotted on cross-section paper and divided into tenths of the depth. All of the curves in which the mean velocity was less than 1.86 feet were combined by combining all the observations at each tenth of depth; all in which the mean velocity was more than 1.86 feet were combined in the same way. From the resultant vertical velocity curves the results in the following table are taken:

Comparison of velocity by double floats and current meters from measurements by T. G. Ellis on Connecticut River in 1874.

Observations.	Mean velocity, in feet per second, at various depths.											Mean.	Varia tion
	Surface.	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	Bottom.		
Meter (av. velocity less than) 1.86.	1.15	1.19	1.22	1.22	1.19	1.14	1.11	1.03	0.94	0.79	0.46	1.063	P. ct.
Float (av. velocity less than) 1.86.	1.50	1.52	1.52	1.53	1.49	1.46	1.35	1.29	1.20	0.97	0.58	1.337	5.
Meter (av. velocity more than) 1.86	2.96	3.08	3.12	3.06	2.99	2.91	2.78	2.62	2.42	2.16	1.47	2.735	
Float (av. velocity more than) 1.86	3.26	3.24	3.17	3.17	3.11	3.01	2.94	2.80	2.60	2.35	1.78	2.891	5.7
All meter observations	2.05	2.12	2.15	2.13	2.08	2.01	1.93	1.81	1.67	1.47	0.96	1.887	
All float observations	2.38	2.38	2.35	2.35	2.30	2.23	2.15	2.04	1.90	1.60	1.18	2.114	12

It is seen from the foregoing table that the velocity as found with the meter is from 6 to 26 per cent less than that found with the floats, and that the difference between the meter and float velocities increases as the velocity decreases.

[a] Rept. Chief Eng. U. S. A., 1878, Appendix B.

Ellis gives also the following values of the ratio of mean velocity in a vertical (V_m) to mid-depth velocity in that vertical ($V_{\frac{1}{2}D}$) and the ratio of distance below the surface of mean velocity (m) to total depth (D).

Mean ratios from vertical velocity curves obtained by Ellis on Connecticut River in 1874.

Observations.	$Vm \div V_{\frac{1}{2}D}$	$m \div D$
Meter at low velocities	0.933	0.656
Floats at low velocities	0.918	0.622
Mean of low velocities	0.930	0.652
Meter at high velocities	0.939	0.628
Floats at high velocities	0.961	0.637
Mean of high velocities	0.944	0.629
Mean of meter measurements	0.937	0.638
Mean of float measurements	0.947	0.632
Mean of all measurements	0.940	0.636

From the foregoing table it is seen that $\frac{V_m}{V_{\frac{1}{2}D}}$ varies from about 0.92 to 0.96 and $\frac{m}{D}$ from 0.62 to 0.66—about the same range of values.

MARR'S EXPERIMENTS.

In October, 1879, G. A. Marr made simultaneous velocity measurements of the Mississippi River at Burlington, Iowa, with double floats like those shown in fig. 1 and with five current meters.[a] The river where measured was about 2,000 feet wide, and the path of the floats was near the center, where the depth was about 16.4 feet. Ten floats were run in succession in a period of about twenty-five minutes over a distance of 200 feet, and the time of entering and leaving each quarter of the run was recorded on a chronograph of eight pens. The five current meters were fastened to a cable held vertically 16 feet below the end of the run. One meter was at mid depth (the depth of the lower float), and the other four were arranged two above and two below it, so as to divide the depth into equal parts. One pen of the chronograph recorded the time of passage of each float, two recorded the time as shown by a chronometer, and the other five recorded the revolutions of the five meters. From a table giving the velocity of each float over each quarter of the run and the velocity shown by each of these meters during the time of passage over these distances the following table has been prepared.

[a] McKenzie, A., Report on Current Meter Observations, Burlington, Iowa, 1884.

Comparison of velocity by double floats and current meters from measurements by G. A. Marr on Mississippi River at Burlington, Iowa, October, 1879.

Number of float.	Mean velocity found from 200-foot run.		Difference ($V_m - V_f$).	Mean velocity found from 50-foot run.		Difference ($V_m - V_f$).
	Float (V_f).	Meter (V_m).		Float (V_f).	Meter (V_m).	
	Ft. per sec.	*Ft. per sec.*	*Ft. per sec.*	*Ft. per sec.*	*Ft. per sec.*	*Ft. per sec.*
1.....	2.543	2.604	+0.061	2.660	2.597	−0.063
2.....	2.741	2.546	−0.195	2.976	2.657	−0.319
3.....	2.634	2.520	−0.114	2.674	2.630	−0.044
4.....	2.446	2.435	−0.011	2.456	2.481	+0.025
5.....	2.850	2.468	−0.382	2.847	2.590	−0.257
6.....	2.672	2.700	+0.028	2.778	2.649	−0.129
7.....	2.514	2.545	+0.031	2.674	2.484	−0.190
8.....	2.832	2.457	−0.375	2.857	2.749	−0.108
9.....	2.476	2.625	+0.149	2.569	2.821	+0.252
10.....	2.829	2.758	−0.071	3.012	2.823	−0.189
	2.654	2.566	.....	2.750	2.648	.....

In the foregoing observations the velocity of a float was not the same over each 50-foot length of the run. Each velocity given in the second column is a mean for the whole run of 200 feet, and the meter velocity in the third column is for the time of the full run. The velocity in the fifth column is for the fourth section of the run, and the corresponding number in that column is the meter velocity for the next consecutive and equal interval of time. The chief thing to be noticed is that the float velocity, whether computed from the whole run of 200 feet or from the shorter run of 50 feet, is greater than the meter velocity by about 3.5 per cent. We may note also that we have here forty mid-depth velocities in nearly the same vertical taken in about twenty-five minutes. For the ten found from the floats on the long run the variations of velocity from the mean are from −7.4 per cent to +7.8 per cent, and for the short run of 50 feet the variations are from −9.5 per cent to +10.7 per cent. For the meter long run the variations are from −7.5 per cent to +12.1 per cent, and for the short run from −6.5 per cent to +6.3 per cent.

Individual measurements of velocity by either of these methods differ much from the mean, but the means by the two methods differ little from each other. The excess of mid-depth velocity by double floats over that by meter may be due to the upper float dragging the lower one.

HENRY'S EXPERIMENTS.

D. F. Henry also gives a comparison of velocity measured with double floats like the one shown in fig. 1 (p. 15) and with current meter, made on St. Clair River in 1869.[a] The results of the comparison are given in the following table:

[a] Jour. Franklin Inst., vol. 62, p. 322.

Comparison of velocity by double floats and current meter from measurements by D. F. Henry on St. Clair River in 1869.

Number of observations.	Direction and velocity of wind.	Depth of observation below surface.	Velocity of current.		
			Floats.	Meter.	Difference.
	Miles per hour.	*Feet.*	*Ft. per sec.*	*Ft. per sec.*	*Ft. per sec.*
50.....	3.26 up	1	3.619	3.655	−0.036
62.....	1.92 up	5	3.759	3.783	−0.024
56.....	1.27 up	10	3.703	3.674	+0.029
50.....	0.48 up	15	3.590	3.516	+0.074
54.....	0.29 down	20	3.598	3.405	+0.193
31.....	0.53 up	25	3.637	3.441	+0.196
37.....	0.80 up	30	3.546	3.279	+0.267
29.....	2.18 down	35	3.556	3.166	+0.390
12.....	4.85 down	40	3.636	3.142	+0.494
7.....	0.74 up	45	3.542	2.985	+0.557

It is seen that the float velocity is less than the meter velocity to a depth of about 5 feet. Below that depth the float velocity is greater than the meter velocity, the difference increasing with the depth. The discharge as found with the floats is 10 per cent greater than that found with the meter. While the action o' the upper float on the lower float may not account for all the difference between the float and meter velocities, it will account for some of it. The velocity as shown by the vertical velocity curve increases from the surface to a depth of from 5 to 10 feet and then decreases to the bottom. The surface float will therefore move more slowly than the lower float for depths of from 0 to 5 or 10 feet, and hence the lower float will be retarded by the upper float to that depth. For depths greater than 10 feet the upper float will move faster than the lower float and will drag it, and the velocity shown will be too large.

Henry gives also a comparison of velocity as found with double floats and current meter in a small canal at Ogdensburg, N. Y.[a] The floats were run over a distance of 200 feet at 3 feet below the surface, the time being recorded on a chronograph. The current meter was held at the same depth at the middle of the run. The results are as follows:

Comparison of velocity by double floats and current meter from measurements made by D. F. Henry in small canal at Ogdensburg, N. Y., 3 feet below surface.

Number of observations.	Velocity.		
	Floats.	Meter.	Difference.
	Ft. per sec.	*Ft. per sec.*	*Ft. per sec.*
24.....	1.992	1.980	−0.012
6.....	1.876	1.916	+0.040
6.....	1.476	1.434	−0.042

These results indicate that although the mean of a few float observations may differ much from the corresponding meter velocity, the means for thirty or forty observations differ little. The meter gives a little less velocity than the floats.

[a] Jour. Franklin Inst. Vol. 62, p. 259.

Henry also gives a comparison of velocity found with floats and a propeller meter in St. Clair River. The lower float was 1 foot below the surface, and the meter was held at the same depth. The mean velocity given by 50 floats was 3.619 feet per second; that by meter was 3.655 feet per second, or 0.036 foot per second greater than by floats. There was a light wind blowing upstream, which may have retarded the floats somewhat.

GORDON'S EXPERIMENTS.

In 1873 Gordon made comparisons of the discharge of the Irawadi River at Saiktha, Burma, as measured with double floats and with current meter. From these he concluded that the double-float discharge was about 10 per cent too large. He therefore reduced all discharges found with double floats by that amount.

FLOAT RODS AND CURRENT METERS.

NEW YORK STATE CANAL SURVEY EXPERIMENTS.

During the progress of the New York State canal survey of 1900 comparative discharge measurements were made on four occasions for the purpose of comparing the discharge of the canal as found by one rod-measuring party with that found by another party using similar rods and methods, and of comparing the discharge found with rods with that found simultaneously with a current meter. The results of the comparisons are summarized in the following table:

Comparison of results of experiments of New York State canal survey during the summer of 1900.

Place.	Date.	V'_r	V''_r	V_m	Q'_r	Q''_r	Q_m	$\frac{Q'_r - Q''_r}{Q'_r}$	$\frac{Q'_r - Q_m}{Q'_r}$	$\frac{Q''_r - Q_m}{Q''_r}$
	1900.	*Ft. per sec.*	*Ft. per sec.*	*Ft. per sec.*	*Cu. ft. per sec.*	*Cu. ft. per sec.*	*Cu. ft. per sec.*	*Per ct.*	*Per ct.*	*Per ct.*
Lockport	Sept. 14	.....	1.440	1.531	.....	805.70	799.85	.....	.....	+0.7
Rochester	Sept. 20	.....	0.704	0.736	.....	235.12	251.57	.....	.....	−7.0
Do	Sept. 21	0.893	0.884	.....	297.82	302.04	.....	−1.41	.....	.....
Do	....do...	.....	0.889	0.929	.....	301.35	310.83	.....	.....	−3.1
Do	....do...	.....	0.899	0.939	.....	307.08	317.54	.....	.....	−3.4
Boonville	Sept. 25	1.073	0.813	.....	227.22	227.51	.....	−0.13	.....	.....
Do	....do...	.....	0.850	1.079	.....	238.81	228.66	.....	.....	+4.2
Do	....do...	.....	0.836	1.111	.....	233.54	234.70	.....	.....	−0.5
Glens Falls	Oct. 8	0.821	0.690	.....	164.73	172.77	161.50	−4.88	+1.94	+6.5
Do	....do...	0.756	0.640	.....	149.66	158.57	155.23	−5.92	−3.72	+2.1

V'_r=mean velocity found by party No. 1, with rods.
V''_r=mean velocity found by party No. 2, with rods.
V_m=mean velocity found with meter, using six-tenths-depth method.
Q'_r=discharge found by party No. 1, with rods.
Q''_r=discharge found by party No. 2, with rods.
Q_m=discharge found with meter, using six-tenths-depth method.

The Lockport experiments were made on the Erie Canal about 1 mile above the locks at Lockport, in a rock cut with rough, nearly vertical sides. The meter measurements were made 700 feet above

the place where the discharge was measured with rods. The observations extended over a period of thirty-four minutes, during which time 60 rods were passed over a run of 21 feet, with an immersion of 80 to 90 per cent of the depth, and 13 meter observations of fifty seconds each were made at six-tenths depth below the surface. There was no measurable fluctuation of surface level during the test.

The Rochester experiments were made on the Rochester Aqueduct, which has smooth and nearly vertical sides. The width was 43 feet and the depth 7.5 feet. The flow was undisturbed for a half mile above the section of measurement, but there was a sharp curve less than 100 feet below the point where the lower rod-measuring party was stationed. In the first comparison of rods with meter, September 20, the period of observation was forty-two minutes; 64 rods were passed over a 20-foot run with 92 per cent depth of immersion; 19 meter observations of fifty seconds each were made by the six-tenths-depth method; the surface level fluctuated 0.05 foot during the observations, and rain interfered somewhat with the work. In the first rod comparison, September 21, the observations extended over a period of sixty-five minutes; 79 rods were passed by party No. 1, with a depth of immersion of from 90 to 97 per cent, and 100 rods by party No. 2, with a depth of immersion of 92 per cent. In the first rod-and-meter comparison of September 21 the period of observation was forty-two minutes; 92 rods were passed by party No. 2, with 90 per cent depth of immersion, and 36 meter observations of fifty seconds each were taken by the six-tenths-depth method. In the second rod-and-meter comparison the period of observation was forty minutes; 80 rods were passed, with 92 per cent depth of immersion, and 36 meter observations of fifty seconds each were made at six-tenths depth.

The Boonville comparisons were made on the Black River Canal at Boonville, where the top width of the canal is 43 feet, with gradual sloping sides, and a maximum depth at center of 7.7 feet. There was an unobstructed flow for at least one-fourth mile above and one-half mile below the place selected. The parties were stationed about 200 feet apart. There was practically no water entering or leaving the canal between these points. In the rod comparison, which lasted one hundred and forty-four minutes, with some interruptions, 185 rods were passed over a 20-foot run by party No. 1, and 143 rods by party No. 2. In the first meter comparison, which lasted fifty-eight minutes, 55 rods were passed and 36 meter observations were made at six-tenths depth. In the second meter comparison, which lasted fifty-three minutes, 77 rods were passed and 42 meter observations of fifty seconds each were made by the six-tenths-depth method.

The Glens Falls comparisons were made on the Glens Falls feeder, near Glens Falls. At the place selected for the tests the feeder is 32 feet wide and 6 feet deep, with nearly vertical sides. There is a bend

in the feeder about 100 feet above the upper place of measurement, and another about 1,000 feet below the lower place of measurement. The meter measurements were made from a bridge about 100 feet from each of the points where the rod measurements were made. The first comparison lasted seventy-six minutes, during which time party No. 1 passed 56 rods, party No. 2 passed 90 rods, and 28 meter observations of fifty seconds each were made. In the second comparison, which lasted fifty-eight minutes, party No. 1 passed 63 rods, party No. 2 passed 108 rods, and 28 meter observations of fifty seconds each were made.

The method of making these comparisons is the same as that used in the canal-discharge measurements, which is described on pages 16, 17.

The Rochester and Glens Falls comparisons are not so satisfactory as those at Lockport and Boonville. In the former the measurements were made too near a curve. The large variation in the time of run of the individual rods indicates this fact, and surface-level fluctuations and passing boats interfered somewhat with the work. There is a possibility of leakage from the feeder between the points of measurement at Glens Falls. No leakage was visible on the surface, but there is a large amount of leakage not far distant from the place of measurement. The measurements there indicate a gradual loss from the upper to the lower point of measurement.

The results of the rod comparisons agree more closely than those of meter with rods. It must not be concluded from the Rochester comparisons that because the rod discharges agree closely and the meter discharge differs from the rod discharge 7 per cent this error is all due to the meter. The two sets of velocity measurements obtained with the meter during this comparison agree closely, whereas the time of run of the individual rods differs considerably. At least half of this difference is due to errors in the rod discharge measurements.

FLOAT RODS AND WEIR.

FRANCIS'S EXPERIMENTS.

In 1856 J. B. Francis compared the discharge of a canal at Lowell, Mass., as found with float rods with that shown by a standard weir. This wooden canal or flume was 27.75 feet wide where the first 5 experiments were made and 14 feet wide where the remaining 5 experiments were made. The length of run was 70 feet, with a starting run of 28 feet. The time of run was obtained to a tenth of a second, and the depth to 0.0001 foot. Every precaution was taken to obtain accurate results. The floats were tin tubes, 2 inches in diameter.

[a] Francis, J. B., Lowell Hydraulic Experiments, p. 170.

loaded at the lower end so as to float upright. They were put in the water at each foot of width of the canal, and the time of run was recorded on a chronograph. The observed velocities were plotted on cross-section paper and a mean curve was drawn among the points showing change in velocity from one side of the canal to the other. The mean velocity in the cross section was found from this curve by dividing the velocity area by the width. The variation of the observed velocities of individual rods from the velocity curve was from 8.57 per cent above to 11.4 per cent below this curve in experiment No. 1. The rods used were not all of the proper depth of immersion, and Francis derived the following formula for computing the correct discharge: $Q''=Q'[1-0.116(\sqrt{D}-0.1)]$, in which Q' is the measured discharge and D the depth of water in the flume less the depth of the immersed part of the rod, divided by the depth of water in the flume. After applying this formula he found that the variation of the rod discharge from the weir discharge for the 115 experiments is as follows: For 9 it is less than 0.1 per cent, for 79 it is less than 1 per cent, for 23 it is from 1 per cent to 1.9 per cent, for 2 it is from 2 per cent to 2.9 per cent, and for 1 it is 3.69 per cent. The mean velocity in these experiments varied from 0.5 foot to 5 feet per second.

CORNELL UNIVERSITY EXPERIMENTS.

At the close of the field work of the New York State canal survey of 1900 a series of experiments was made at the hydraulic laboratory of Cornell University for the purpose of comparing discharge as found with rods and current meter on the canals with that shown by the Cornell standard weir. These experiments were planned by the consulting engineer of the New York State canal survey, Mr. E. Kuichling, and were carried out under the direction of Prof. G. S. Williams, in charge of the hydraulic laboratory, by the writer and by Mr. W. P. Boright, C. E., assisted by members of the class in hydraulics.

The surface level of the sections of the canals fluctuates rapidly on account of lockage of boats and irregular feed of water to lower sections. It was therefore necessary to make discharge measurements rapidly and to make short runs and starting runs. The depth of water in the different sections varied from 5 to 10 feet at the center, so that the immersed portion of the rods used varied from 75 per cent to 95 per cent of the depth of the water. The mean velocity varied from nearly 2 feet per second west of Lockport to 0 on the summit levels. In the laboratory experiments the conditions were duplicated as closely as possible and the work was performed in the same way as on the canals, so that the results of the comparisons give a fair idea of the accuracy of the discharge measurements of the State canals and feeders.

Pl. II shows the laboratory canal and the observers employed in

making the tests. A description of the laboratory and of the method of obtaining standard weir discharge will be found on pages 60 to 65. The gates at the head of the canal were opened until the head on the weir was such as to give the desired discharge; then the slits in the bulkhead and the height of the gates at the lower end of the canal were adjusted so as to give the desired depth of water in the channel. As soon as the flow became steady three parties began measuring the discharge of the canal, one with rods and two with current meters. Two series of rods were used, one of 75 per cent depth of immersion, the other of 90 per cent depth of immersion, the former lettered, the latter numbered, so that they could easily be recognized as they passed under each wire. The lengths of the run and of the starting run were the same as in the canal work, being from 25 to 7 feet for the former and from 10 to 8 feet for the latter, depending on the velocity. The number of rods used in an experiment varied from 50 to 150. Meter party No. 1 measured the discharge by the six-tenths-depth and the integration methods, and then assisted the rod-measuring party, four stop watches being used and the number of rods run in a given time nearly doubled. During each experiment the head on the weir, the depth of water in the canal, and the slope of the surface were read every thirty seconds and were checked occasionally by readings of the hook gage. The time of run of each rod was platted on squared paper, using distance from side of canal as abscissæ and time of run, in seconds, as ordinates, and a mean time curve was drawn among them. The length of run divided by the mean ordinate of this curve gave the mean velocity, which multiplied by the cross-sectional area gave the discharge. The results of this rod-and-weir discharge comparison are given in the following table:

HYDRAULIC LABORATORY OF CORNELL UNIVERSITY, SHOWING CORPS ENGAGED IN MAKING DISCHARGE EXPERIMENTS

Comparison of float-rod and weir velocity and discharge measurements at Cornell hydraulic laboratory.

Date.	Discharge over standard weir per second.	Rod gagings in laboratory canal.						Error of discharge referred to standard weir.		Mean error of discharge referred to standard weir.	
		Velocity from weir discharge per second.	Depth of water in canal.	Immersion, 75 per cent.		Immersion, 90 per cent.					
				Velocity per second.	Discharge per second.	Velocity per second.	Discharge per second.	75 per cent immersion.	90 per cent immersion.	75 per cent immersion.	90 per cent immersion.
1900.	*Cu. ft.*	*Feet.*	*Feet.*	*Feet.*	*Cu. ft.*	*Feet.*	*Cu. ft.*	*Per ct.*	*Per ct.*	*P. ct.*	*P. ct.*
October 24	214.588	1.428	9.356	1.422	213.674	.....	.....	+0.43	.....		
Do	140.514	0.945	9.253	0.962	142.966	0.949	141.084	−1.74	−0.37	1.32	0.75
Do	71.670	0.484	9.222	0.490	72.621	0.476	70.546	−1.33	+1.57		
Do	38.308	0.269	8.789	0.276	38.990	0.272	38.425	−1.78	−0.31		
October 26	178.553	1.473	7.543	1.510	183.185	1.525	184.832	−2.59	−3.52		
Do	125.298	1.033	7.549	1.080	131.000	1.058	128.332	−4.55	−2.42	3.96	2.09
October 27	61.272	0.510	7.471	0.530	63.658	0.520	62.457	−3.89	−1.93		
Do	30.089	0.254	7.376	0.266	31.543	0.255	30.239	−4.83	−0.50		
Do	196.387	1.983	6.163	2.067	204.695	2.044	202.407	−4.23	−3.06		
Do	132.408	1.308	6.299	1.363	137.956	1.341	135.730	−4.19	−2.51	4.22	2.95
Do	91.056	0.910	6.228	0.963	95.372	0.944	94.471	−4.74	−3.64		
Do	50.309	0.535	5.847	0.555	52.176	0.549	51.612	−3.71	−2.59		
October 31	198.258	1.475	8.365	1.537	206.554	1.507	202.523	−4.18	−2.15		
Do	125.299	0.930	8.393	0.958	129.174	.....	.....	−3.09	.....	4.68	2.78
October 29	65.287	0.486	8.355	0.512	68.766	0.501	67.288	−5.33	−3.07		
November 1	31.140	0.235	8.255	0.249	33.043	0.242	32.114	−6.11	−3.13		
										3.54	2.14

It is seen that the discharge obtained with the rods is larger for all depths, velocities, and percentage depths of immersion than the corresponding weir discharge, except in two cases. The greatest variation from the weir discharge is 6.11 per cent for the 75 per cent depth of immersion and 3.64 per cent for the 90 per cent depth of immersion. This variation from the weir discharge, or error, increases as the depth, velocity, and depth of immersion decrease. The mean error for all depths and velocities is 3.54 per cent for the 75 per cent depth of immersion and 2.14 per cent for the 90 per cent depth of immersion. The greatest variations from these means are +1.14 per cent and −2.22 per cent for the 75 per cent depth of immersion and +0.81 per cent and −1.39 per cent for the 90 per cent depth of immersion, a range of 3.4 per cent in the former and 2.2 per cent in the latter. From the last two columns is seen the extent to which the error increases as the depth decreases.

It is to be expected that the rod discharge will be greater than that of the weir, especially for the 75 per cent depth of immersion, for as the rods do not extend to the bottom they are not affected by the bottom layers of slowly moving water, and hence move faster than the mean velocity. By applying Francis's correction formula (p. 55) to these results others will be obtained agreeing more closely with those shown by the weir. It will be shown further on (p. 93) that the weir discharges used in these experiments and computed from Bazin's formula are probably too small by from 0.5 per cent to 2.5 per cent,

the former for a head on weir of 1 foot, the latter for a head of 2.5 feet. When these two sets of corrections are applied it will be found that the rod discharge is correct to within about 2 per cent. This degree of accuracy is only attained, however, by many observations, equivalent to six or twelve gagings, with the rods started 2 feet apart from side to side of canal.[a]

CURRENT METER AND WEIR.

FTELEY AND STEARNS'S EXPERIMENTS.

Fteley and Stearns have made some comparisons of the discharge of the Sudbury conduit as obtained by weir measurement and by current meter.[b] The weir was of the sharp-crested type, 19 feet long, and was located at the head of the conduit. The flow to it was controlled by rectangular gates, which were changed slightly from time to time so as to keep the head on the weir constant. The current-meter measurements were made from a manhole in the conduit 6,000 feet below the weir. The meter used was a Fteley like that shown in fig. 5, p. 22. Two methods of operating the meter were employed, viz, the point and the integration. When the point method was used the meter was held for thirty seconds at several points in the cross-sectional area, depending on the depth. For the greatest depth of 4.5 feet the number of points of observation was 167. The rate of moving the meter while integrating was from 0.1 foot to 1 foot per second. The results for the 8-vane meter are as follows:

Comparison of discharge measured with 8-vane Fteley meter and over weir.

No. of experiment.	Number of comparisons included in experiment.	Rate of integration.	Depth of center of meter from surface.	Ratio of speed of meter to speed of water.	Mean velocity of water.	Variation of meter discharge from weir discharge.
			Feet		*Ft. per sec.*	*Per cent.*
1	1	0	1.5	0	1.83	+1.2
2	2	0	2.0	0	2.13	0.0
3	1	0	3.0	0	2.56	+0.1
4	1	0	4.0	0	2.83	+0.1
5	1	0	4.5	0	2.93	−0.4
6	1	0	3.0	0	1.71	−0.2
7	3	0.1	1.5	0.055	1.83	+0.8
8	3	0.1	2.0	0.047	2.13	+0.1
9	8	0.1	2.5	0.042	2.37	−0.6
10	3	0.1	3.0	0.039	2.56	−0.2
11	6	0.1	3.5	0.037	2.71	−0.1
12	4	0.1	4.0	0.035	2.83	−0.5
13	4	0.1	4.5	0.034	2.93	−0.8
14	4	0.1	2.5	0.042	2.37	+0.2
15	16	0.5	2.5	0.211	2.37	−3.6
16	15	0.5	3.5	0.185	2.71	−1.8
17	20	1.0	3.5	0.369	2.71	−6.5
18	3	0.1	3.0	0.058	1.71	−1.0
19	4	0.5	3.0	0.292	1.71	−4.5
20	7	1.0	3.0	0.585	1.71	−9.4

[a] See also pp. 90–93.

[b] Trans. Am. Soc. Civ. Eng., vol. 12, p. 301.

The results of experiments Nos. 2 to 6 show a remarkably close agreement between measurements by this meter and those over the weir. The length of an observation, thirty seconds, was too short to give a good average indication of the velocity. The discharge varied somewhat during the experiment. The depth of water in the conduit being small, any error in the depth must have introduced considerable error in the result. We would not expect, therefore, so close an agreement, which appears all the more remarkable when it is seen from the following table that the discharge shown by two weirs may differ by 3 per cent:

Values of m in formula $Q = mlh\sqrt{2gh}$, as found by Bazin and by Fteley and Stearns.[a]

Head on weir (h).	Values of m.		$m_1 - m_2$	$\frac{m_1 - m_2}{m_1}$
	Fteley and Stearns (m_2).	Bazin (m_1).		
Meters.	*Meters.*	*Meters.*	*Meters.*	*Per cent.*
0.1424	0.4149	0.4290	0.0141	3.29
0.1969	0.4166	0.4263	0.0097	2.28
0.2497	0.4191	0.4259	0.0068	1.60
0.3008	0.4218	0.4266	0.0048	1.12
0.3530	0.4247	0.4276	0.0029	0.68
0.3957	0.4272	0.4285	0.0013	0.33
0.4434	0.4296	0.4295	0.0001	0.02
0.4888	0.4323	0.4310	−0.0013	−0.33

[a] Annales des ponts et chaussées, mémoires et documents, 1888, p. 416.

It is seen that for heads on the weir of from 0.5 foot to 1 foot the variation in m is from 3 to 1 per cent.

EXPERIMENTS AT CORNELL UNIVERSITY HYDRAULIC LABORATORY.

INTRODUCTION.

The results of numerous weir, current-meter, and other experiments by the writer at the Cornell hydraulic laboratory are discussed in detail on the following pages; also the application of the results to ordinary river gaging and the general conclusions reached from these recent studies and investigations.

The investigation was begun by the writer in May, 1900, while a fellow in civil engineering at Cornell University. It was interrupted from July 1 to December 4 of that year, on account of the New York State canal survey, in which the writer participated, and again from December 9 to April 1, on account of freezing weather, and was completed in May, 1901. Its object was to compare the discharge of the Cornell University experiment canal as obtained with a sharp-crested weir with that obtained from current meters operated by different methods, and thus determine the accuracy of velocity and discharge

obtained by current-meter methods. The scarcity of meters available for the work, the lack of ready facilities for frequent testing and rating of the meters used, and the scarcity of water during the first months of the investigation render the results somewhat less valuable than they otherwise would be.

The writer takes this occasion to acknowledge his indebtedness to Mr. F. H. Newell, chief hydrographer of the United States Geological Survey, for his hearty cooperation; to Prof. E. A. Fuertes, director and dean of the College of Civil Engineering, Cornell University, for the use of the hydraulic laboratory and a chronograph, computing machine, current meter, and electric register, and for kindly interest in the work; to Prof. Gardener S. Williams, in charge of the hydraulic laboratory, for many valuable suggestions and for the use of a chart for converting head on the Cornell University standard weir into cubic meters; also to Mr. C. E. Torrance, scholar in civil engineering, to members of the senior class in civil engineering, and to Mr. C. D. Cass, assistant mechanician of the college, for their cheerful assistance.

DESCRIPTION OF CORNELL HYDRAULIC LABORATORY.

The Cornell hydraulic laboratory consists essentially, as shown in Pls. II, III, and IV and fig. 15, of a canal 16 feet wide, 10 feet deep, and 415 feet long, with concrete bottom and sides and with a bed slope of 1 foot in 500. There are six rectangular gates at the upper end for controlling water entering the canal and four gates near the lower end, two on each side, for controlling the depth of water in the canal. Fifty-nine feet below the entrance gates is a bulkhead about 10.5 feet high built of 12-inch by 12-inch timbers. On the top of this, or 11.1 feet above the bottom of the canal, is fastened a steel angle iron 3.5 inches by 5 inches by 16 feet long, with edges five-sixteenths inch thick, which forms the standard sharp-crested weir. The lower end of the canal is closed with 12-inch by 12-inch timbers, which can be moved up or down if desired, so as to allow the water to pass out of the canal at the end instead of under the side gates. In the upper chamber are two sets of baffles for checking the velocity of the water after it passes under the entrance gates. One of these is 7.4 feet from the gates and is made of 4-inch by 12-inch timbers placed horizontally, with the wide face toward the current and spaced from 8 to 12 inches apart. On the upper side of these timbers are 1-inch by 6-inch strips, fastened diagonally and spaced 12 inches from center to center. The second baffle is about 6.5 feet below the first one and is similar in construction, except that the 1-inch by 6-inch strips are placed vertically and are spaced 10 inches from center to center. Twenty-eight feet below the weir is a baffle made of 1-inch by 6-inch strips placed 3 inches from center to center, with the narrow side

FLUME, PENSTOCK, AND WHEEL HOUSE OF CORNELL HYDRAULIC LABORATORY CANAL

toward the current. To still further quiet the waters after passing the weir, when the depth is small and the velocity large, a second baffle was constructed of 1-inch by 6-inch pieces placed vertically, spaced 2 inches from center to center, and fastened on the upper side of the baffle just described.

Air is admitted under the falling sheet of water through 12-inch by 10-inch passages in the concrete, one on each side of the weir. The head of the weir and the depth of water in the canal are measured by piezometers and a hook gage. As rough checks on these readings gage boards and a dial gage were occasionally read. The piezometer for giving the head on the standard weir is formed of three 1-inch pipes (P, P, P, fig. 15) perforated for 3 feet of their length with $\frac{1}{4}$-inch holes 6 inches apart around the pipes. These are placed 27 feet upstream from the weir, with their axes parallel with the axis of the

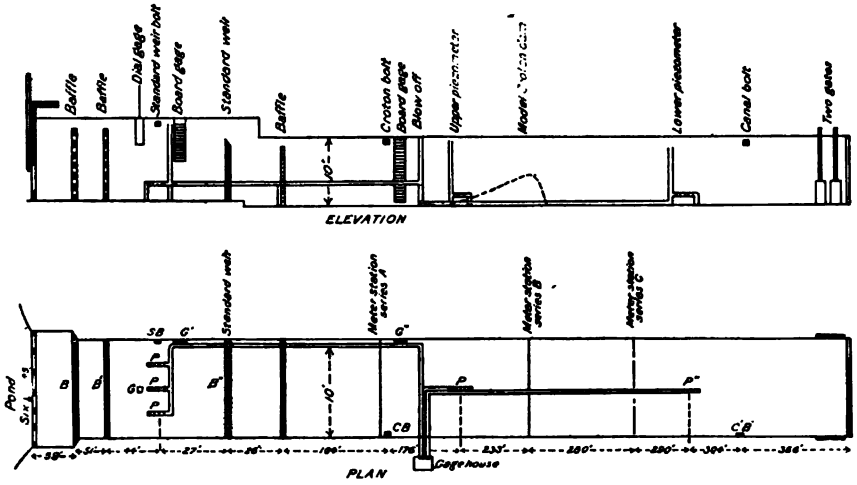

FIG. 15.—Plan and elevation of Cornell University experiment canal.

canal, 4 feet from the bottom, and each connected to a pipe which extends down the south side of the canal to a point opposite the gage house. From this point it is carried across the canal along the bottom and up into the gage house and is connected to a glass tube by a rubber hose. The tube is fastened to a scale marked to 2 mm. spaces. The canal piezometers are similar to those of the weir, but have only one entrance pipe, which is in the center of the canal. The surface elevation of the water was read with the hook gage at the three points in fig. 15 marked "Standard weir bolt," "Croton bolt," and "Canal bolt," the elevation of each of which is known. The dial gage and the board gage G' show approximately the head on the standard weir in feet and hundredths and enable the man operating the gates to keep a nearly constant head. A board gage (G'') opposite the "Croton bolt" (CB) shows approximately the depth of water in the canal.

SCOPE OF EXPERIMENTS.

The experiments naturally divide themselves into five groups, which we have called series A, B, C, D, and E. The experiments of series A were made at a section 158 feet below the standard weir and 75 feet upstream from a model of the Croton dam the height of which was 6 feet and the upstream face of which had a gradual slope and extended to within 29 feet of the meter station. This model dam formed a pool above it at the meter station and gave a zero velocity for 6 feet depth of water. The range of depths and velocities was therefore small. The water surface during the experiments was smooth, without boiling or swirls, with most of the foam made by the water falling over the weir moving down in two parallel lines 3 or 4 feet from each side. Three current meters were used in this series of experiments, viz, large Price meter No. 88, small Price meter No. 351, and the Cornell University Fteley meter. The latter was always held with a rod, the small Price meter with a cable, and the large Price meter with a cable in some of the experiments and with a rod in other experiments. About 3¼ inches of mud had accumulated on the bottom of the canal above the model dam. After the first two experiments the mud was removed for a space of 4 or 5 feet above and below the meter station. For all other experiments the bottom was clean and smooth.

The experiments of series B were made 234 feet below the standard weir after the model dam had been removed. They were with high velocities and shallow depths, the scarcity of water at the time restricting the depth somewhat. In all these experiments except Nos. 6 and 8 the tail gates were open, so that there was no backwater. The surface of the water was quite rough, being a succession of waves, without swirls or boiling. (See Pl. IV.) The same three meters were used as in series A. The Fteley and the large Price were held with rods—the small Price with a rod in some of the experiments and with a cable in other experiments.

The experiments of series C were made 280 feet below the standard weir, as part of the tests of accuracy of the discharge measurements of the New York State canals and feeders made during the summer of 1900. They cover a range of depths from 6 to 9.5 feet and velocities from 0.23 foot to 2 feet per second. In most of the experiments of this series nearly all of the water was passed out of the canal through horizontal slits in the bulkhead. The water surface was comparatively smooth, being a succession of long waves, without boiling or swirls. Two meters were used—small Price No. 363, operated by the six-tenths-depth and integration methods, and small Price No. 351, operated by the ordinary method. The former was held with a cable, the latter with a rod.

The experiments of series D were made 220 feet below the standard weir, with Haskell meter No. 3 and small Price meter No. 363, each

CORNELL HYDRAULIC LABORATORY CANAL, SHOWING DISTURBED WATER SURFACE DURING EXPERIMENTS IN SERIES B.

held with a cable. The range of depth was from 5 to 9.5 feet, and the velocities from 0.25 foot to 2 feet. The water surface was like that in series C—long waves, without boiling or swirls. Three methods of obtaining velocity were used—the ordinary, the six-tenths, and the integration. This series is not complete, the work being stopped by the freezing of the canal. The piezometers could not be used on account of the freezing of the pipes. The hook gage only was used to measure the head on the weir and the depth of water in the canal.

The experiments of series E were made with two small Price meters and a Haskell meter, held with cables at station 220 feet below the standard weir. The depth in this series varied from 4.6 feet to 9.3 feet, and the mean velocity from 1.5 feet to 3 feet per second. It was designed to extend the results of the previous year to the higher velocities. The discharge measurements were made in pairs, as in series D, so as to compare the results of the meters with each other as well as the results of each meter with the corresponding weir discharge. The head on the weir and the elevation of water surface were obtained from readings of the portable hook gage. The water surface was quite rough for these higher velocities, being a succession of waves, without swirls. To quiet it and enable the surface elevation to be more accurately measured, in experiments 9 to 22 two planks, 2 inches by 12 inches by 15 feet long, were placed across the canal about 3 feet apart and 154 feet from the weir, fastened at the ends so as to float on the surface and break the force of the waves. In experiments 23 to 50 three of these planks were used. The planks served their purpose very well, but distorted somewhat the vertical velocity curves.

METHOD OF EXPERIMENTATION.

The first operation in each experiment was to adjust the entrance gates to admit such a quantity of water that with the desired depth of water at the meter station the velocity would be about the desired rate. The tail gates were then opened or closed somewhat, as the case required, until the water reached the desired depth and remained steady. The standard weir gage was then read every thirty seconds and the canal gages (one or both) were read every five minutes during the time the meter observations were being taken. When the hook gage was used it was read six times in succession, as quickly as possible, at each of the three points in fig. 15 marked "Standard bolt," "Croton bolt," and "Canal bolt," already described. When the ordinary method of operating the meter was used the meter was held at from two to six points, depending on the depth, in each of from five to eight verticals, and the number of the revolutions of the meter wheel in from one to two periods of from thirty to sixty seconds each was obtained. Usually it was two periods of fifty seconds each. In series C, ordinary method, one period of sixty seconds was used and the revolutions were

read on a recorder. In all the other experiments, except those with the Fteley meter, the revolutions were obtained from indications of an electric buzzer. When the meter was operated by the six-tenths-depth method it was held at about six-tenths depth below the surface in eight verticals, viz, 1, 3, 5, 7, 9, 11, 13, and 15 feet from the south side of the canal, and the revolutions were obtained for two periods of fifty seconds each. The meter was then held at the same depth in the same verticals in the inverse order, i. e., 15, 13, etc., 1 foot from the south side of the canal, and the revolutions were again counted for two periods of fifty seconds each. When the integration method was used in series C the meter was moved in the following way: It was started at 1 foot from the bottom (center of meter 0.5 foot above bottom) and 1 foot from the side of the canal, and was passed slowly to the surface at 4.5 feet from the side, then to the bottom at 8 feet from the side, then to the surface at 11.5 feet from the side, and then to the bottom at 15 feet from the side. The time at the beginning and end was noted and the total number of revolutions were counted. The meter was then carried back over the same path at about the same speed, and the time of passage and the number of revolutions were obtained as before. The mean time and the mean number of revolutions of these two passages across the canal are used as a single experiment. When this method was used in series A the meter was carried only once across the canal, and it was started and stopped at the bottom (center of meter 0.5 foot above bottom) and close to each side, and reached the surface 4 feet and 12 feet from the south side of the canal. When this method was used in series E the meter was moved slowly from the surface to the bottom (center of meter from 2.5 inches below surface to 5.5 inches above bottom) and back to the surface in each of the eight verticals in which the meter was held when the ordinary point method was used. The time of passage of the meter down and back again in each vertical was noted and the revolutions of the meter wheel were counted.

REDUCTION OF EXPERIMENTS.

The standard weir gage readings were averaged for the time of each discharge measurement, and the reading for zero head subtracted, giving the head on the weir in double centimeters. By means of a chart for converting head on weir into cubic meters per second and a computing machine the discharge in cubic feet per second was easily obtained. The depth of water in the canal was found by adding to the mean canal gage reading in feet the difference in elevation of the canal bottom and the zero of the scale. The discharge when the ordinary method was used was computed from the vertical velocity curve for that experiment (see figs. 23 to 28, pp. 75–77). The observations were usually taken in the eight verticals at 1, 3, 5, 7, 9, 11, 13, and 15 feet from the south side of the canal, and at the same distance above the

bottom. When observations in any vertical were missing, as in verticals 7 and 11 in series C, they were supplied by interpolation. The mean number of revolutions at the depth of each observation for these eight verticals was then found, converted into velocity by the rating table, plotted (as in figs. 23 to 28), and a mean curve drawn. The mean abscissa of this curve is the mean velocity in the cross section; the discharge is the product of this velocity and the cross-sectional area.

When the six-tenths-depth method was used a horizontal curve of velocity was drawn in a similar manner and the mean velocity was found from it. The mean velocity by the integration method was obtained by dividing the revolutions by the corresponding time and converting the quotient into velocity.

DISCUSSION OF RESULTS.

COMPARISONS OF DISCHARGE MEASUREMENTS.

The following tables give the results of the discharge comparisons of series A to D. The standard-weir discharge (third column) is the mean for the time over which the meter observations (ordinary method) extended. It is assumed to be the same for the shorter period of measuring the discharge by the six-tenths-depth and the integration methods. This is not strictly true for the larger discharges, but it is near enough for practical purposes. The true velocity is obtained by dividing the weir discharge by the cross-sectional area at the meter station. The other columns are self-explanatory.

Comparisons of discharge over Cornell University standard weir with measurements by current meters.

SERIES A.

Date.	No. of experiment.	Standard weir discharge (Q).	Meter observations.								Variation.	
			Depth of water in canal at meter section.	True velocity (V) at meter section.	Kind of meter.	Number of observations.	Ordinary method.		Integration method.		$\frac{Q-Q_o}{Q}$	$\frac{Q-Q_1}{Q}$
							V_o	Q_1	V_1	Q_1		
		Cu. ft. per sec.	*Feet.*	*Feet p. sec.*	(a)		*Feet p. sec.*	*Cu. ft. per sec.*	*Feet p. sec.*	*Cu. ft. per sec.*	*Per ct.*	*Per ct.*
1900.												
May 5...	1	147.991	7.996	1.166	L. P. 88..	76	1.097	138.478	.....	.....	+ 6.43	.....
Do....	2	65.024	7.167	0.573	....do ...	29	0.447	50.714	.....	.....	+22.01	.....
May 7...	3	78.481	7.258	0.683	Fteley ..	31	0.677	77.780	.....	.....	+ 0.90	.....
May 8..	4	162.684	8.104	1.268	S. P. 351.	38	1.267	162.461	1.252	160.62	+ 0.14	+1.26
Do....	5	100.379	7.542	0.841	....do ...	20	0.790	94.318	.....	.....	+ 6.03	.....
May 9...	6	193.624	8.369	1.462	....do ...	35	1.435	190.109	.....	.....	+ 1.81	.....
Do....	7	131.178	7.845	1.056	....do ...	18	1.036	128.657	1.031	128.04	+ 1.92	+2.32
Do....	8	131.037	7.848	1.056	L. P. 88..	18	0.982	122.000	1.006	124.93	+ 6.90	+4.63
Do....	9	131.072	7.845	1.055	Fteley ..	18	1.148	142.570	.....	.....	− 8.77	.....
May 15..	10	148.556	7.984	1.175	....do ...	40	1.286	162.540	.....	.....	− 9.43	.....
Do....	11	119.947	7.729	0.980	S. P. 351.	32	0.939	114.887	0.978	119.66	+ 4.22	+0.26
May 16..	12	74.137	7.281	0.643	L. P. 88..	36	0.533	61.432	.....	.....	+17.14	.....
Do....	13	93.457	7.484	0.788	....do ...	32	0.744	88.110	.....	.....	+ 5.72	.....
Do....	14	74.137	7.280	0.643	Fteley ..	29	0.642	73.985	.....	.....	+ 0.20	.....
Do....	15	93.386	7.484	0.788	....do ...	32	0.834	98.804	.....	.....	− 5.80	.....

(a) L. P.=large Price; S. P.=small Price. The figures given are the meter numbers.

Comparisons of discharge over Cornell University standard weir, etc.—Continued.

SERIES B.

Date.	No. of experiment.	Standard weir discharge (Q).	Meter observations.						Variation.
			Depth of water in canal at meter section.	True velocity (V) at meter section.	Kind of meter.	Number of observations.	Ordinary method.		
							V_o	Q_o	
1900.		*Cu. ft. per sec.*	*Feet per sec.*	*Feet per sec.*	(a)		*Feet per sec.*	*Cu. ft. per sec.*	*Per ct*
May 19	1	50.366	1.115	2.823	S. P. 351.	16	2.790	49.774	1.11
Do	2	50.260	1.115	2.823	Fteley ..	16	3.599	64.206	—27.75
Do	3	50.154	1.110	2.824	L. P. 88.	16	3.103	53.790	— 7.26
May 21	4	65.625	1.299	3.157	Fteley ..	24	3.344	69.502	— 5.91
Do	5	65.625	1.299	3.157	L. P. 88.	24	3.085	64.118	+ 2.30
Do	6	66.896	2.000	2.091	Fteley ..	24	2.500	80.000	—18.56
Do	7	21.616	0.786	1.719	L. P. 88.	16	1.538	19.341	+ 10.54
Do	8	66.731	2.029	2.056	....do ...	24	1.900	61.682	+ 9.07
Do	9	33.060	0.741	2.766	Fteley ..	16	3.273	32.296	+ 2.30
May 22	10	21.369	0.550	2.428	....do ...	16	2.945	25.916	—21.28
Do	11	94.410	1.885	3.130	....do. ..	24	3.789	114.250	—21.01
Do	12	94.410	1.885	3.130	L. P. 88.	24	3.102	93.556	+ 0.90
Do	13	57.854	1.198	3.018	Fteley ..	24	3.720	71.305	—23.27
Do	14	57.854	1.198	3.018	L. P. 88.	24	2.947	56.589	+ 2.18
Do	15	43.761	0.953	2.870	S. P. 351.	16	2.698	43.055	+ 1.62
Do	16	43.761	0.953	2.870	Fteley ..	16	3.474	52.944	—20.99
Do	17	29.598	0.693	2.671	S. P. 351.	16	2.425	26.876	+ 9.19
Do	18	29.598	0.693	2.671	Fteley ..	16	3.187	34.768	—17.47
Do	19	18.982	0.464	2.553	S. P. 351.	8	2.290	16.983	—10.30
Do	20	18.932	0.464	2.553	Fteley ..	8	2.932	21.744	—14.84

a L. P.=large Price; S. P.=small Price. The figures given are the meter numbers.

Comparison of discharge over Cornell University standard weir with measurements by current meters.

SERIES C.

Date.	Number of experiment.	Standard weir discharge (Q).	Meter observations.							Variation.		
			Depth of water in canal at meter section.	True velocity (V) at meter section.	Small Price No. 351, ordinary method.		Small Price No. 363.			Small Price No. 351. $\left(\frac{Q-Q_0}{Q}\right)$.	Small Price No. 363.	
					V_0	Q_0	Ordinary method.		Integration method (Q_1).		$\frac{Q-Q_6}{Q}$	$\frac{Q-Q_1}{Q}$
							V_6	Q_6				
1900.		*Cu. ft. per sec.*	*Feet.*	*Ft. per sec.*	*Ft. per sec.*	*Cu. ft. per sec.*	*Ft. per sec.*	*Cu. ft. per sec.*	*Cu. ft. per sec.*	*Per cent.*	*Per cent.*	*Per cent.*
October 24	1	216.131	9.460	1.409	1.446	219.836	1.465	222.980	220.393	− 1.77	−4.11	− 0.63
Do	2	140.515	9.374	0.932	0.969	146.060	0.985	148.528	148.099	− 4.13	−6.13	− 5.37
Do	3	71.681	9.338	0.477	0.493	73.050	0.487	73.167	71.514	− 2.38	−2.08	− 1.59
Do	4	38.254	8.910	0.265	0.259	37.420	0.278	40.113	38.330	+ 2.03	−6.00	−16.16
October 26	5	178.553	7.673	1.444	1.449	179.130	1.489	184.075	174.679	− 0.33	−3.09	− 5.17
Do	6	125.298	7.640	1.018	0.999	122.974	1.047	128.880	120.789	+ 1.85	−2.78	− 5.12
October 27	7	61.272	7.582	0.502	.....	.....	0.522	63.771	61.327	.....	−4.88	− 4.08
Do	8	31.176	7.557	0.255	0.215	26.180	0.258	31.488	28.070	+16.01	−1.04	−22.10
Do	9	50.310	6.006	0.515	0.494	48.208	0.540	52.696	46.451	+ 4.17	−5.92	− 7.65
Do	10	196.387	6.278	1.940	1.890	192.799	1.989	201.367	188.232	+ 1.83	−1.46	− 4.86
Do	11	132.414	6.399	1.283	1.251	129.078	1.318	135.991	123.816	+ 2.52	−3.18	− 4.27
Do	12	91.056	6.319	0.893	0.847	86.318	0.920	93.758	88.872	+ 5.20	−5.08	− 2.96
October 29	13	65.319	8.476	0.478	0.464	68.325	0.492	67.146	60.459	+ 3.07	−2.35	− 2.80
October 31	14	198.258	8.470	1.454	1.460	199.115	1.514	206.480	187.795	− 0.44	−3.71	− 6.76
Do	15	125.301	8.508	0.915	0.890	121.921	0.949	130.001	114.522	+ 2.69	−3.75	− 9.54
November 1	16	31.223	8.455	0.229	0.201	27.364	0.240	32.674	25.186	+12.37	−2.47	−39.30

Comparison of discharge over Cornell University standard weir with measurements by current meters—Continued.

SERIES D.

Date.	Number of experiment.	Standard weir discharge (Q).	Meter observations.							Variation.		
			Depth of water in canal at meter section.	True velocity (V) at meter section.	Kind of meter.[a]	Ordinary method.		Six-tenths depth method (Q_6).	Integration method (Q_1).	$\frac{Q-Q_o}{Q}$	$\frac{Q-Q_6}{Q}$	$\frac{Q-Q_1}{Q}$
						V_o	Q_o					
1900.		*Cu. ft. per sec.*	*Feet.*	*Ft. per sec.*		*Ft. per sec.*	*Cu. ft. per sec.*	*Cu. ft. per sec.*	*Cu. ft. per sec.*	*Per cent.*	*Per cent.*	*Per cent.*
December 6	1	233.262	9.505	1.528	S. P. 363	1.519	231.976		235.082	+0.55		−0.77
Do	2	233.262	9.505	1.528	Hask. 3	1.484	226.380		239.968	+2.95		−2.88
December 7	3	229.731	9.317	1.535	do			239.120	235.828		− 4.09	−2.66
Do	4	117.673	[b] 9.536	0.768	do	0.832	[b] 127.421	[b] 126.498		−8.28	− 7.50	
Do	5	117.673	[b] 9.536	0.768	S. P. 363	0.794	[b] 121.601	[b] 119.301		−3.34	− 1.40	
Do	6	95.176	[b] 9.524	0.622	Hask. 3	0.676	[b] 103.401	[b] 105.846		−8.63	−11.21	
Do	7	95.176	[b] 9.524		S. P. 363	0.563	[b] 99.883	[b] 97.892		−4.93	− 2.84	
December 8	8	221.749	7.245	1.905	Hask. 3	1.864	216.932	218.680	228.340	+2.13	+ 1.38	−3.08
Do	9	221.749	7.245	1.905	S. P. 363	1.867	217.281	222.521		+2.01	− 0.35	
Do	10	191.165	7.217	1.649	Hask. 3	1.675	194.183	200.209	202.647	−1.58	− 4.73	−6.00
Do	11	191.165	7.217	1.649	S. P. 363	1.651	191.400	192.793		+0.20	− 0.84	
Do	12	110.186	7.183	0.955	Hask. 3	1.022	117.919	118.802	119.609	−7.01	− 7.84	−8.55
Do	13	110.186		0.955	S. P. 363	0.955	110.188	110.858			- 0.68	
Do	14	56.717	7.113	0.496	Hask. 3			58.241			− 6.14	
					S. P. 363			58.270			− 2.73	
Do	15	57.071	5.154	0.689	Hask. 3	0.730	60.455	62.277	62.277	−5.94	− 9.10	−9.10
Do	16	57.071	5.154	0.689	S. P. 363	0.672	55.651	57.308		+2.49	− 0.42	

[a] S. P. = small Price. Hask. = Haskell. The figures given are the meter numbers.
[b] Hook gage broken; depth not accurately measured.

It is seen from the foregoing tables that the meter discharge differs from the weir discharge all the way from 0 to 40 per cent. One might say, without careful examination, that the current meter must be a very unreliable velocity-measuring instrument; but when we examine these results carefully and plot them according to velocity we find the case very different. Take first the results by the ordinary method. These are plotted for each meter in fig. 16, using mean velocity as abscissæ and percentage difference as ordinates. It is seen that nearly all of these points fall on well-defined lines. The few exceptions are those in which the depth is small and the errors of depth and position of meter have a large effect on the measured discharge. The discharge obtained with the Fteley meter differs more from the corresponding weir discharge than that obtained with the other meters. This is because it had never been rated. The constant we have used in reducing the observations with this meter—viz, 1 rev-

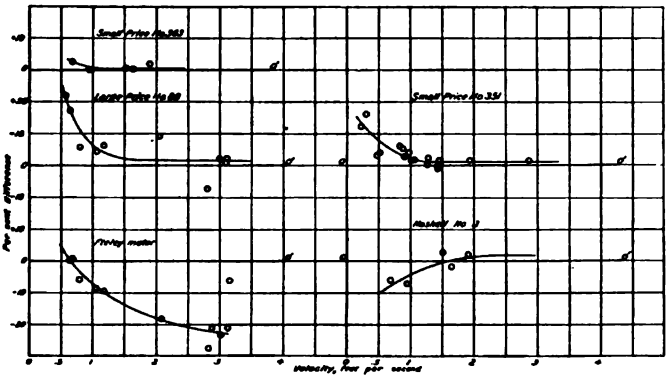

FIG. 16.—Diagram showing difference between meter discharge by ordinary method and weir discharge, from measurements in Cornell University experiment canal. Line 0-0′ represents weir discharge.

olution per second at 1 foot velocity=1.072—was obtained from observations by students. It is not, however, much in error for this velocity, but is largely in error for other velocities. The rating obtained from these weir experiments is given in the table on page 81. For the other three meters the percentage error is small for velocities above 1 foot per second. For the Price meters the error is positive and increases rapidly as the velocity decreases. For the Haskell meter the error is negative and increases, negatively, rapidly as the velocity decreases. The error in rating, if we may call it such, is not so large as it appears from these diagrams, for a 10 per cent error in a small quantity is only a 1 per cent error in a quantity ten times as large. The rating tables as derived from these curves and as found by moving the meter through still water are given in the table on page 81. Experiments with depths of 0.75 foot or less (Nos. 9 and 17 to 20 of series B) are not plotted in the diagram (fig. 16), as the prob-

able error is large; nor are experiments 1 to 4 of series C, because the upper and lower parts of the vertical curves are not well defined; nor

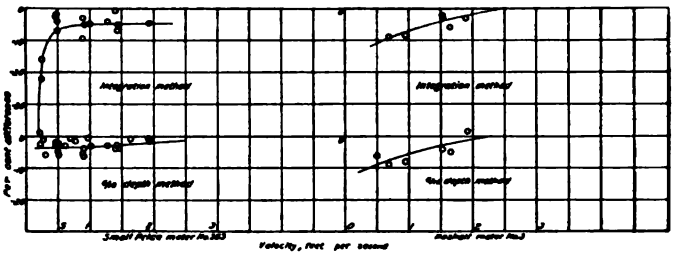

FIG. 17.—Diagram showing difference between meter discharge by six-tenths depth and integration methods and weir discharge, from measurements in Cornell University experiment canal. Line 0-0′ represents weir discharge.

experiments 4 to 7 of series D, because there appears to be an error in the hook-gage work of those experiments.

In fig. 17 are plotted the percentage differences between weir discharge and discharge obtained by the six-tenths depth and the integration methods. It is seen that the meter discharge by these methods is greater than the weir discharge, being 4 or 5 per cent greater for velocities above 1 foot per second. In the integration method, for small velocities the difference increases very rapidly as the velocity decreases. It is seen from figs. 16 and 17 that while the ordinary method gives discharge 1 or 2 per cent less than the weir for velocities above 1 foot per second, the six-tenths depth and the integration methods give discharge 4 or 5 per cent greater than the weir.

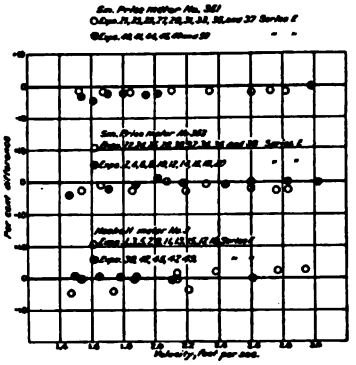

FIG. 18.—Diagram showing difference between meter discharge with different meters and weir discharge in experiments of series E in Cornell University experiment canal. Zero line indicates weir discharge.

The results of the discharge measurements of series E are given in the following table and are plotted in fig. 18, which shows three sets of double comparisons. Experiments 1 to 20 give a comparison of simultaneous measurements with small Price meter No. 363 and Haskell meter No. 3, and also with the discharge obtained at the same time with the weir. Experiments 21 to 38 give similar comparisons for the two small Price meters and the weir, and experiments 39 to 50 give similar comparisons for Haskell and Price meters and the weir.

Comparison of discharge over Cornell University standard weir with measurements by current meters.

SERIES E.

No. of experiment.	Date.	Depth of water in canal at meter section.	True velocity at meter section.	Standard weir discharge (Q).	Meter discharge (Q_m).	Variation $\left(\frac{Q-Q_m}{Q_m}\right)$.	Kind of meter.[a]	Difference in meter discharges.
	1901.	*Feet.*	*Ft. per sec.*	*Cu. ft. per sec.*	*Cu. ft. per sec.*	*Per cent.*		*Per cent.*
1	April 1....	9.075	1.472	204.41	214.08	−4.73	Hask. 3....	0.86
2	.....do....	9.075	1.456	204.41	212.33	−3.87	S. P. 363..	
3	April 2....	6.079	2.208	208.88	216.28	−3.54	Hask. 3....	0.99
4	.....do....	6.079	2.187	208.88	214.22	−2.55	S. P. 363..	
5	.....do....	7.453	1.744	200.81	208.85	−4.00	Hask. 3....	2.03
6	.....do....	7.453	1.710	200.81	204.77	−1.97	S. P. 363..	
7	.....do....	5.742	2.137	197.64	197.02	−0.31	Hask. 3....	1.58
8	.....do....	5.742	2.171	197.64	200.15	+1.27	S. P. 363..	
9	April 5....	7.409	1.869	222.67	222.68	0.00	Hask. 3....	0.75
10	.....do....	7.409	1.883	222.67	224.35	−0.75	S. P. 363..	
11	.....do....	6.295	2.137	220.33	216.00	+1.96	Hask. 3....	1.82
12	.....do....	6.295	2.177	220.33	220.03	+0.14	S. P. 363..	
13	.....do....	5.628	2.377	219.38	214.82	+2.08	Hask. 3....	2.51
14	.....do....	5.628	2.438	219.38	220.33	−0.43	S. P. 363..	
15	.....do....	5.294	2.611	221.96	222.02	−0.03	Hask. 3....	0.34
16	.....do....	5.294	2.602	221.96	221.27	+0.31	S. P. 363..	
17	.....do....	4.892	2.764	222.81	217.32	+2.46	Hask. 3....	2.22
18	.....do....	4.892	2.827	222.81	222.27	+0.24	S. P. 363..	
19	.....do....	4.617	2.986	224.75	218.32	+2.86	Hask. 3....	2.85
20	.....do....	4.617	3.019	224.75	224.50	+0.01	S. P. 363..	
								1.60
21	April 6....	4.808	2.985	230.36	230.66	−0.13	S. P. 351..	0.71
22	.....do....	4.808	3.006	230.36	232.30	−0.84	S. P. 363..	
23	April 10...	8.744	1.670	230.75	234.68	−1.70	S. P. 351..	0.71
24	.....do....	8.744	1.657	230.75	232.85	−0.99	S. P. 363..	
25	.....do....	7.874	1.846	229.28	233.61	−1.89	S. P. 351..	0.88
26	.....do....	7.874	1.862	229.28	235.64	−2.77	S. P. 363..	
27	.....do....	6.889	2.108	229.91	233.25	−1.45	S. P. 351..	1.68
28	.....do....	6.889	2.073	229.91	229.38	+0.23	S. P. 363..	
29	April 11...	6.129	2.340	227.16	230.25	−1.36	S. P. 351..	0.86
30	.....do....	6.129	2.320	227.16	228.28	−0.50	S. P. 363..	
31	.....do....	5.483	2.596	225.01	228.59	−1.60	S. P. 351..	0.31
32	.....do....	5.483	2.604	225.01	229.30	−1.91	S. P. 363..	
33	.....do....	5.185	2.725	224.06	226.98	−1.30	S. P. 351..	1.23
34	.....do....	5.185	2.758	224.06	229.73	−2.53	S. P. 363..	
35	.....do....	5.003	2.816	222.60	226.37	−1.69	S. P. 351..	0.65
36	.....do....	5.003	2.834	222.60	227.82	−2.34	S. P. 363..	
37	.....do....	9.256	1.522	223.40	226.38	−1.33	S. P. 351..	1.05
38	.....do....	9.256	1.538	223.40	228.71	−2.38	S. P. 363..	
								0.90
39	April 12...	9.344	1.486	224.63	223.11	+0.68	Hask. 3....	3.68
40	.....do....	9.344	1.541	224.63	231.37	−3.00	S. P. 351..	
41	.....do....	8.934	1.613	221.39	231.58	−4.60	.....do....	4.67
42	.....do....	8.934	1.543	221.39	221.53	+0.07	Hask. 3....	
43	.....do....	8.297	1.646	221.39	219.50	+0.81	.....do....	3.21
44	.....do....	8.297	1.700	221.39	226.70	−2.40	S. P. 351..	
45	.....do....	7.806	1.775	221.49	222.69	+0.54	Hask. 3....	2.61
46	.....do....	7.806	1.803	221.49	226.19	−2.07	S. P. 351..	
47	.....do....	7.266	1.879	221.21	219.34	+0.85	Hask. 3....	3.38
48	.....do....	7.266	1.943	221.21	226.81	−2.53	S. P. 351..	
49	.....do....	6.439	2.132	221.03	220.43	−0.27	Hask. 3....	1.80
50	.....do....	6.439	2.182	221.03	225.61	−2.07	S. P. 351..	
								3.22

[a] S. P.=small Price. Hask.=Haskell. The figures given are the meter numbers.

The principal results of the foregoing comparisons have been condensed in the following table:

Table showing percentage variation between weir discharge and meter discharge, and between meter discharge by different instruments of experiments in Series E.

Experiments.	Kind of meter.	Difference between standard weir discharge and meter discharge.				Difference in discharge of simultaneous meter measurements.			
		Max.	Min.	Mean.[a]	Range.	Max.	Min.	Mean.	Range.
		Per ct.	*Per ct.*	*Per ct.*	*Per cent.*	*Per ct.*	*Per ct.*	*Per ct.*	*Per ct*
1 to 20	Haskell No. 3...	−4.73	0.00	−0.33	−4.73 to +2.86	.....	.....	.....	.....
Do.....	S. Price No. 363.	−3.87	+0.01	−0.76	−3.87 to +1.27	2.85	0.34	[a] 0.29 [b] 1.60	2.51
21 to 38	S. Price No. 351.	−1.89	−0.13	−1.38	−1.89 to −0.13	.....	.....	.....	.....
Do.....	S. Price No. 363.	−2.77	+0.23	−1.56	−2.77 to +0.23	1.68	0.31	[a] 0.18 [b] 0.90	1.37
39 to 50	S. Price No. 351.	−4.60	−2.07	−2.78	−4.60 to −2.07	.....	.....	.....	.....
Do.....	Haskell No. 3...	+0.85	−0.27	+0.40	+0.85 to −0.27	4.67	1.80	[a] 3.22 [b] 3.22	2.87

[a] Regarding signs. [b] Disregarding signs.

It is seen from this that in none of these fifty experiments does a meter discharge differ from a corresponding weir discharge 5 per cent, and in no case do two simultaneous meter discharges differ from each other 5 per cent. In experiments 1 to 20 the mean variation from the weir discharge is −0.33 per cent for the Haskell meter and −0.76 per cent for the Price meter, while the mean difference between corresponding meter discharges is 0.29 per cent. In experiments 21 to 38 the mean variation from the weir discharge is 1.38 per cent for Price meter No. 351 and −1.56 per cent for Price meter No. 363, while the mean difference between corresponding meter discharges is 0.18 per cent. In experiments 39 to 50 the mean variation from the weir discharge is 2.78 per cent for the Price meter and 0.40 per cent for the Haskell meter, while the mean difference between corresponding meter discharges is 3.22 per cent.

The range of variation of meter discharge from weir discharge is greater than that of corresponding meter discharges. This is to be expected, however, since there are a number of possible errors in the former which are not in the latter. Errors in measuring head, depth, calibration of weir, velocity, and rating of meter affect the former, while only the two latter errors affect corresponding meter discharges.

It is also seen that the mean of the difference between simultaneous discharge measurements Nos. 21 to 38 is only 0.18 per cent, and the extreme range in these eighteen consecutive measurements is only 1.37 per cent. How much of this error is due to errors in the rating table and how much to errors in observing velocity we were unable to determine, as good facilities for rating were not at hand. We are, however, justified by these experiments in the statement that the discharge of this canal can, with a small Price meter, be measured to within less than 1 per cent when the mean velocity in the canal is from 1.5 feet to 3 feet per second.

VELOCITY CURVES.

Curves of equal velocity in the cross-sectional area of observation are shown in figs. 19, 20, 21, and 22. Four experiments are selected (one from each series) from the sixty-five experiments of series A to D by the ordinary method, to show the variation of velocity in the sections. They are self-explanatory. Fig. 20 illustrates a case of small depth, high velocity, and no back-water effect. The maximum velocity is at the surface, nearly midway between the banks. In figs.

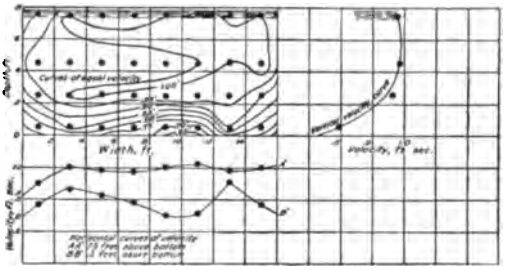

FIG. 19.—Curves of equal velocity in experiment No. 11, series A.

19, 21, and 22 the maximum velocity is below the surface and not midway between the banks. It is less at the center than 3 or 4 feet from either side. This is true of all the experiments of series A. We tried admitting the water to the upper chamber of the canal in various ways, by opening some of the gates and closing or partly closing others, but the distribution of velocity appeared to be nearly inde-

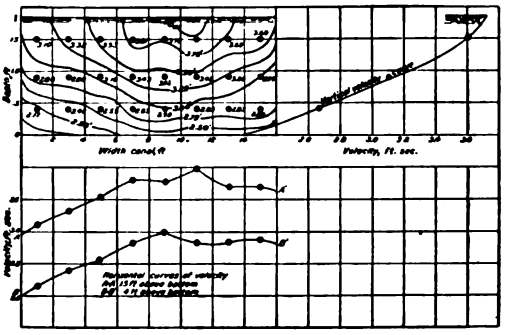

FIG. 20.—Curves of equal velocity in experiment No. 12, series B, with large Price meter No. 88.

pendent of the openings through which the water was admitted to the canal.

Horizontal velocity curves are shown in these same figures (19 to 22), the velocity curve at six-tenths depth below the surface being shown, and also the curves for surface and bottom. In these figures the velocity near the north side of the canal is somewhat less than that near the south side. This is true of all the experiments of series C

and D. It is due to the north side of the canal being much rougher than the south side, the concrete of the north side having been cut away preparatory to its repair just previous to making the experiments of series C.

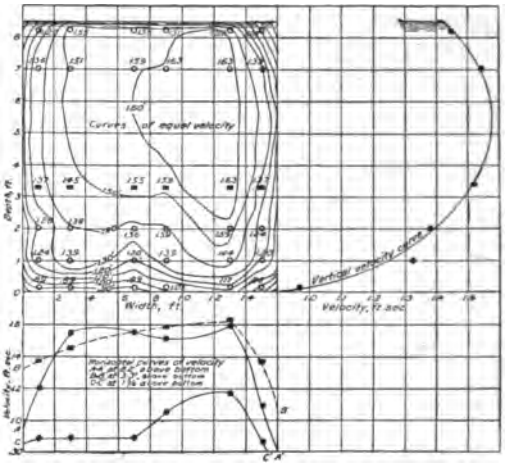

FIG. 21.—Curves of equal velocity in experiment No. 14, series C. The dots are for small Price meter No. 351; heavy black squares for small Price meter No. 363.

Vertical velocity curves for these four series of experiments (A to D) are shown in figs. 23 to 28. Each of these curves is the mean of observations in the eight verticals 1, 3, 5, 7, 9, 11, 13, and 15 feet from the south side of the canal. For series C and D, figs. 26, 27, and 28, the observations obtained with small Price meter No. 363 at six-tenths

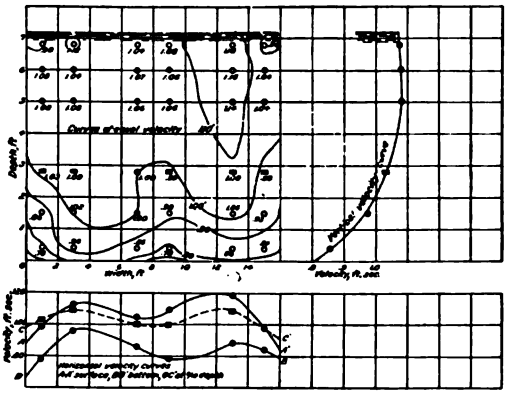

FIG. 22.—Curves of equal velocity in experiment No. 20, series D. The dots are for Haskell meter No. 3; heavy black squares for small Price meter No. 363.

depth are shown by the little squares. As a rule they fall outside the curve, meter No. 363 indicating a somewhat greater velocity than meter No. 351. This fact is also shown by the corresponding curves of figs. 16 and 17. The two sets of velocity curves (full and dotted) of

fig. 28 were found from simultaneous velocity measurements. Two parties measured velocity in the same section at the same time. One party, with the Haskell meter, started near the south side of the canal and worked toward the north side; the other party, with the Price meter, started at the north side and worked toward the south side. It is seen from these curves (fig. 28) that the bottom velocities agree more closely than those at the surface, and that the Haskell meter

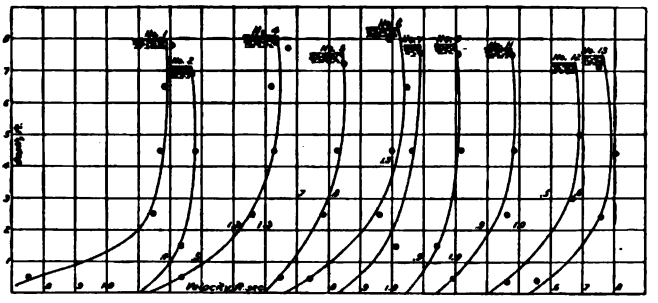

FIG. 23.—Vertical velocity curves obtained with current meters in experiments in series A. Nos. 1, 2, 8, 12, and 13 are for large Price meter No. 88; Nos. 4, 5, 6, 7, and 11 are for small Price meter No. 351.

gives a much larger surface velocity than the Price meter. We should expect a little difference, since the revolving head of the Haskell meter is larger than the wheel of the Price meter, and its center being farther below the surface it must revolve faster. But the difference in velocity is much greater than this little difference in depth of axis would indicate. (See p. 91.)

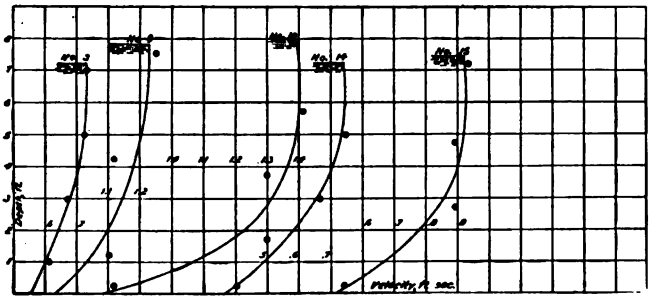

FIG. 24.—Vertical velocity curves obtained with Fteley current meter in experiments in series A.

The shape of these curves evidently depends on the depth and velocity. For depths not exceeding 1 foot it is a nearly straight line, with the maximum velocity at the surface and the mean velocity at mid depth. For depths of 8 or 9 feet and small velocities the curve is very flat, with the maximum velocity at two-tenths to three-tenths depth and the mean velocity at sixty-five-hundredths to seven-tenths depth below the surface. The radius of curvature decreases as the velocity increases. In all of these curves the least velocity is at the

bottom. Nos. 6 and 8, series B, fig. 25, show the effect on the vertical velocity curve of checking the discharge at the lower end of the canal. For all the other curves of fig. 25 there is free discharge. The effect of checking the discharge at the lower end is to force the thread of maximum velocity from the surface to a depth of two-tenths or more. The table on page 77 gives the position of the threads of maximum and

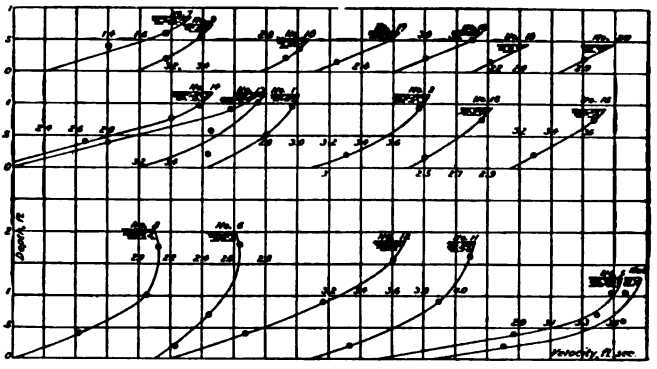

FIG. 25.—Vertical velocity curves obtained with current meters in experiments in series B. Nos. 1, 15, 17, and 19 are for small Price meter No. 351; Nos. 3, 5, 7, 8, 12, and 14 are for large Price meter No. 88; Nos. 2, 4, 6, 9, 10, 11, 13, 16, 18, and 20 are for Fteley meter.

mean velocity taken from the curves shown in figs. 23 to 28. It is difficult to locate the threads with accuracy, since very little change in the shape of the curve would make a great change in the position of the threads. It is easily seen that the shape may be changed somewhat, and yet fit the observations nearly as well as the one drawn and give a discharge differing from it but little. For velocities of a foot

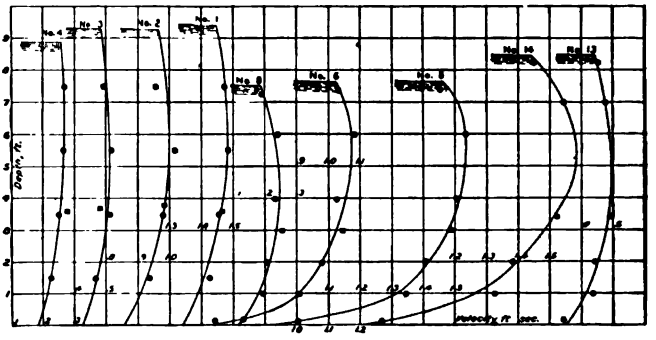

FIG. 26.—Vertical velocity curves obtained with current meters in experiments in series C. Heavy round dots are for small Price meter No. 351; the square blocks are for small Price meter No. 353.

or more per second, however, the threads can be located with a fair degree of accuracy. The position of the thread of maximum velocity as found from series A is eighteen-hundredths depth below the surface; for series B it is one-tenth depth; for series C, thirty-eight-hundredths depth; and for series D, twenty-four-hundredths depth. For the mean

of C and D, in which the conditions are nearly the same, it is thirty-one-hundredths depth. The position of the thread of mean velocity as shown by this table is sixty-six-hundredths depth for series A, fifty-four-hundredths depth for series B, and sixty-six-hundredths

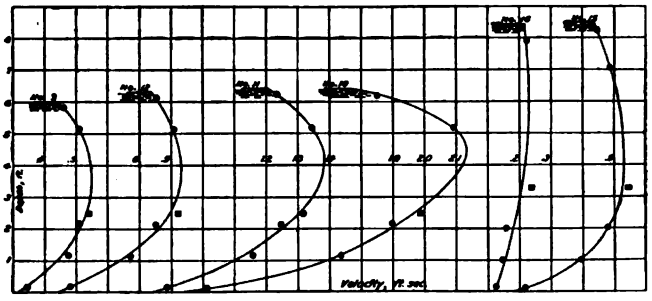

FIG. 27.—Vertical velocity curves obtained with current meters in experiments in series C. Heavy round dots are for small Price meter No. 351; the square blocks are for small Price meter No. 363.

depth for series C and D. It is seen that the threads of mean and maximum velocity are nearer the surface in series A than in series C or D, although the depths do not differ much. This is probably due to the influence of the Croton model dam, the long upstream slope of

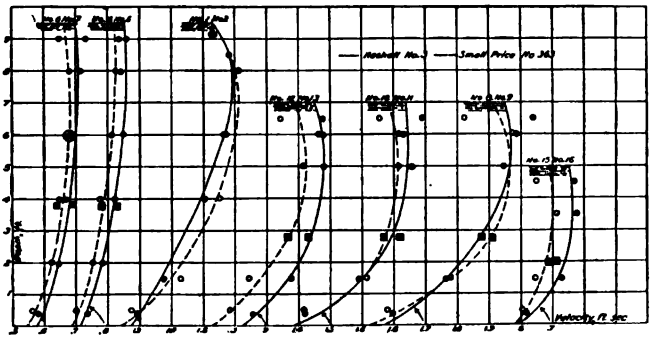

FIG. 28.—Vertical velocity curves obtained with Haskell meter No. 3 and small Price meter No. 363 in experiments in series D. Open circles are for small Price meter No. 363, point method; heavy black dots are for Haskell meter No. 3, point method; heavy black squares are for small Price meter No. 363, six-tenths-depth method.

which has had the effect of forcing the thread of maximum velocity up to thirteen-hundredths depth. As the ratio of width to depth changed from 16 to 2 the thread of mean velocity moved from fifty-four-hundredths depth to sixty-six-hundredths depth below the surface.

Table showing position of threads of maximum and mean velocity in Cornell University experiment canal as found from current-meter discharge measurements made in May, November, and December, 1900.

No. of experiment.	Series A.				Series B.				Series C.				Series D.			
	Velocity (V).	Depth (D).	Ratio $\left(\frac{d_m}{D}\right)$.	Ratio $\left(\frac{d'}{D}\right)$.	Velocity (V_o).	Depth (D).	Ratio $\left(\frac{d_m}{D}\right)$.	Ratio $\left(\frac{d'}{D}\right)$.	Velocity (V_o).	Depth (D).	Ratio $\left(\frac{d_m}{D}\right)$.	Ratio $\left(\frac{d'}{D}\right)$.	Velocity (V_o).	Depth (D).	Ratio $\left(\frac{d_m}{D}\right)$.	Ratio $\left(\frac{d'}{D}\right)$.
	Feet per second.	*Feet.*			*Feet per second.*	*Feet.*			*Feet per second.*	*Feet.*			*Feet per second.*	*Feet.*		
1	1.17	8.00	0.81	0.35	2.79	1.12	1.00	0.40	1.45	9.46	0.65	0.31	1.52	9.51	0.75	0.29
2	0.57	7.17	0.77	0.29	3.60	1.12	1.00	0.49	0.97	9.37	0.61	0.32	1.48	9.51	0.80	0.47
3	0.68	7.26	0.92	0.28	3.10	1.11	1.00	0.49	0.49	9.34	0.52	0.27	.....	9.32	.....	.....
4	1.27	8.10	0.89	0.29	3.34	1.30	1.00	0.42	0.26	8.92	0.62	0.28	0.83	9.54	0.79	0.42
5	0.84	7.54	0.80	0.49	3.09	1.30	1.00	0.38	1.45	7.67	0.70	0.33	0.79	9.54	0.84	0.42
6	1.46	8.37	0.75	0.30	2.50	2.00	0.85	0.44	1.00	7.64	0.72	0.33	0.68	9.52	0.77	0.39
7	1.06	7.85	0.84	0.45	1.54	0.79	1.00	0.45	.....	7.58	.....	.....	0.65	9.52	0.80	0.34
8	1.06	7.85	0.75	0.31	1.90	2.03	0.85	0.37	0.22	7.56	0.60	0.28	1.86	7.25	0.82	0.36
9	1.06	7.85	0.95	0.43	3.27	0.75	1.00	0.47	0.49	6.01	0.60	0.28	1.87	7.25	0.69	0.32
10	1.18	7.98	0.79	0.37	2.95	0.55	1.00	0.50	1.89	6.28	0.71	0.32	1.68	7.22	0.77	0.34
11	0.98	7.73	0.71	0.36	3.79	1.89	1.00	0.40	1.25	6.40	0.66	0.32	1.65	7.22	0.72	0.33
12	0.64	7.28	0.75	0.30	3.10	1.89	1.00	0.43	0.85	6.32	0.60	0.31	1.02	7.18	0.72	0.33
13	0.79	7.48	0.65	0.21	3.72	1.20	1.00	0.46	0.46	8.48	0.56	0.27	0.96	7.18	0.70	0.36
14	0.64	7.28	0.86	0.39	2.95	1.20	1.00	0.49	1.46	8.47	0.65	0.31	.....	7.11	.....	.....
15	0.79	7.48	0.84	0.33	2.70	0.95	1.00	0.48	0.89	8.51	0.53	0.27	0.73	5.15	0.70	0.33
16	.....	.....	.....	.....	3.47	0.95	1.00	0.48	.....	.....	0.623	0.30	0.67	5.15	0.70	0.44
17	.....	.....	.....	.....	2.43	0.69	1.00	0.50	.....	.....	0.755	0.37	.....	.....	.....	.....
18	.....	.....	.....	.....	3.14	0.69	1.00	0.50	.....	.....	.....	.....	.....	.....	.....	.....
19	.....	.....	.....	.....	2.29	0.46	1.00	0.50	.....	.....	.....	.....	.....	.....	.....	.....
20	.....	.....	.....	.....	2.93	0.46	1.00	0.50	.....	.....	.....	.....	.....	.....	.....	.....
	.....	.....	0.815	0.343	.....	.....	0.985	0.457	.....	.....	0.69	0.34	.....	.....	0.755	0.37

V = velocity found from the weir.
V_o = velocity found with meter by ordinary method.

d_m = distance above bottom to thread of maximum velocity.
d' = distance above bottom to thread of mean velocity.

The first of the following tables gives the mean, maximum, and bottom velocities in the experiments of series A, C, and D, also the ratio of bottom velocity to mean velocity and of mean velocity to mid-depth velocity. It is seen that the ratio of mean velocity to mid-depth velocity is much more nearly constant, and hence is better to use than that of mean velocity to bottom volocity. The second table gives the ratio of mean velocity to three-tenths-depth velocity, and of mean velocity to six-tenths-depth velocity in series A, C, and D. The ratio at three-tenths depth is seen to be more nearly constant than that at six-tenths depth.

Velocities at various depths and velocity ratios in Cornell University experiment canal as found from standard weir and current-meter discharge measurements.

No. of experiment.	Series A. Depth (D) in feet.	Velocities in feet per sec.[a] V	V_m	V_b(b)	$V_{\frac{1}{4}D}$	Ratios. $\frac{V_b}{V}$	$\frac{V}{V_{\frac{1}{4}D}}$	Series C. Depth (D) in feet.	Velocities in feet per sec.[a] V	V_m	V_b(b)	$V_{\frac{1}{4}D}$	Ratios. $\frac{V_b}{V}$	$\frac{V}{V_{\frac{1}{4}D}}$	Series D. Depth (D) in feet.	Velocities in feet per sec.[a] V	V_m	V_b(b)	$V_{\frac{1}{4}D}$	Ratios. $\frac{V_b}{V}$	$V+V_{\frac{1}{4}D}$
1	8.00	1.17	1.26	0.80	1.24	0.69	0.90	9.46	1.41	1.45	1.06	1.44	0.74	0.98	9.51	1.58	1.62	1.29	1.57	0.84	0.97
2	7.17	0.57	0.59	0.50	0.56	0.88	0.98	9.37	0.93	0.96	0.83	0.95	0.89	0.98	9.51	1.53	1.64	1.33	1.56	0.87	0.98
3	7.26	0.68	0.76	0.60	0.70	0.88	0.97	9.34	0.48	0.50	0.43	0.50	0.89	0.96	9.32	1.54	.....	.....	.....	.....	.....
4	8.10	1.27	1.37	1.01	1.33	0.80	0.96	8.92	0.27	0.29	0.22	0.29	0.82	0.94	9.54	0.77	.....	.....	.....	.....	.....
5	7.54	0.84	0.87	0.64	0.85	0.76	0.99	7.67	1.44	1.54	1.12	1.52	0.78	0.95	9.54	0.77	.....	.....	.....	.....	.....
6	8.37	1.46	1.55	1.22	1.53	0.82	0.96	7.64	1.02	1.09	0.78	1.06	0.76	0.96	9.52	0.62	.....	.....	.....	.....	.....
7	7.85	1.06	1.10	0.92	1.08	0.87	0.98	7.58	0.59	.....	.....	.....	.....	.....	9.52	0.62	.....	.....	.....	.....	.....
8	7.85	1.06	1.09	0.92	1.07	0.87	0.99	7.56	0.26	0.29	0.15	0.28	0.58	0.93	7.25	1.91	2.01	1.65	1.96	0.86	0.97
9	7.85	1.06	1.10	0.92	1.07	0.87	0.99	6.01	0.52	0.58	0.37	0.57	0.71	0.91	7.25	1.91	2.01	1.61	1.98	0.84	0.96
10	7.98	1.18	1.24	0.83	1.20	0.70	0.98	6.28	1.94	2.17	1.42	2.07	0.73	0.94	7.22	1.65	1.77	1.42	1.69	0.85	0.98
11	7.73	0.98	1.01	0.79	1.00	0.81	0.98	6.40	1.28	1.41	0.94	1.37	0.73	0.94	7.22	1.65	1.72	1.44	1.70	0.87	0.97
12	7.28	0.64	0.71	0.49	0.70	0.77	0.92	6.32	0.89	0.98	0.63	0.96	0.71	0.93	7.18	0.96	1.01	0.80	0.99	0.83	0.97
13	7.48	0.97	0.89	0.65	0.88	0.83	0.90	8.48	0.48	0.54	0.40	0.53	0.79	0.91	7.18	0.96	1.03	0.76	1.00	0.80	0.96
14	7.28	0.64	0.73	0.42	0.68	0.66	0.94	8.47	1.45	1.60	1.04	1.58	0.72	0.92	7.11	.....	.....	.....	.....	.....	.....
15	7.48	0.79	0.87	0.56	0.85	0.71	0.93	8.51	0.92	0.95	0.67	0.95	0.73	0.97	5.15	0.69	0.71	0.59	0.68	0.86	0.99
16	.....	.....	.....	.....	.....	.....	.....	8.46	0.28	0.26	0.15	.....	0.65	.....	5.15	0.69	0.72	0.62	0.68	0.89	0.99
	.....	.....	.....	.....	.....	0.795	0.958	.....	.....	.....	.....	.....	0.747	0.944	.....	.....	.....	.....	.....	0.851	0.974

[a] Correct velocities found from standard weir. [b] Velocity 2½ inches above the bottom.

NOTE.—V=mean velocity; V_m=maximum velocity; V_b=bottom velocity; $V_{\frac{1}{4}D}$=mid-depth velocity.

Velocity ratios in Cornell University experiment canal as found with current meters in May, November, and December, 1900.

No. of experiment.	Series A.				Series C.				Series D.			
	Velocity (v_o).	Depth (D).	Ratio $\left(\frac{v_3}{v_o}\right)$.	Ratio $\left(\frac{v_6}{v_o}\right)$.	Velocity (v_o).	Depth (D).	Ratio $\left(\frac{v_3}{v_o}\right)$.	Ratio $\left(\frac{v_6}{v_o}\right)$.	Velocity (v_o).	Depth (D).	Ratio $\left(\frac{v_3}{v_o}\right)$.	Ratio $\left(\frac{v}{v_o}\right)$.
	Ft. per sec.	*Feet.*			*Ft. per sec.*	*Feet.*			*Ft. per sec.*	*Feet.*		
1	1.10	8.00	1.082	1.050	1.45	9.46	1.022	1.009	1.52	9.51	1.059	1.01
2	0.45	7.17	1.062	1.036	0.97	9.37	1.029	1.017	1.48	9.51	1.071	1.02
3	0.68	7.26	1.066	0.997	0.49	9.34	1.037	1.022	.....	9.32	.....	.....
4	1.27	8.10	1.060	1.014	0.26	8.92	1.031	0.961	0.83	9.54	1.031	0.98
5	0.79	7.54	1.073	1.010	1.45	7.67	1.057	1.021	0.79	9.54	1.039	1.00
6	1.44	8.37	1.070	1.033	1.00	7.64	1.073	1.025	0.68	9.52	1.034	1.00
7	1.04	7.85	1.040	1.008	0.50	7.58	.....	.....	0.65	9.52	1.046	1.01
8	0.98	7.85	1.034	1.013	0.22	7.56	1.091	1.082	1.86	7.25	1.058	1.01
9	1.15	7.86	1.055	1.000	0.49	6.01	1.091	1.069	1.87	7.25	1.056	1.02
10	1.29	7.98	1.083	1.025	1.89	6.28	1.127	1.042	1.68	7.22	1.038	1.00
11	0.94	7.73	1.050	1.021	1.25	6.40	1.094	1.045	1.65	7.22	1.039	1.00
12	0.53	7.28	1.103	1.043	0.85	6.32	1.090	1.056	1.02	7.18	1.060	1.02
13	0.74	7.48	1.070	1.046	0.46	8.48	1.056	1.043	0.96	7.18	1.070	1.01
14	0.64	7.28	1.146	1.001	1.46	8.47	1.081	1.036	.....	7.11	.....	.....
15	0.83	7.48	1.155	1.030	0.89	8.51	1.025	1.042	0.73	5.15	1.041	1.01
16	.....	.....	.....	.....	0.20	8.46	1.139	1.030	0.67	5.15	1.045	1.01
			1.077	1.021			1.069	1.035			1.048	1.01

v_o = velocity obtained by ordinary method.
v_3 and v_6 = velocity at three-tenths and at six-tenths depth below the surface.

MEAN VELOCITY BY INTEGRATION IN VERTICALS.

The mean velocity in the discharge section was obtained by the method of integration in the eight verticals 1, 3, 5, etc., immediately after making experiments 1, 3, 9, and 37 of series E. In the following table the velocity thus obtained is compared with that obtained by the ordinary point method in the experiment just preceding:

Comparison of mean velocity obtained by the ordinary point method and by integration in verticals in Cornell University experiment canal.

Date.	Depth.	Kind of meter.	Velocity.		$\frac{V_p - V_i}{V_p}$
			Point method (V_p).	Integration method (V_i).	
1901.	*Feet.*		*Ft. per sec.*	*Ft. per sec.*	*Per cent.*
April 1	9.08	Haskell No. 3	1.468	1.460	.5
April 2	6.08	do	2.208	2.225	.8
April 5	7.41	do	1.883	1.921	2
April 11	9.26	S. Price No. 351	1.522	1.568	3

It will be seen that the agreement is close, the greatest difference being 3 per cent.

RATING METERS IN STILL AND IN MOVING WATER.

The results of meter and standard-weir comparisons are given in the tables on pages 65 to 68. With a few exceptions, already mentioned, these are shown graphically in figs. 16, 17, and 18, pages 69 and 70. The straight lines 0–0′ represent in each case the weir discharge and the curved lines drawn through the plotted points repre-

sent the percentage variation of the meter discharge from the corresponding weir discharge. The meter discharge is computed from a rating table prepared from ratings in still water. If we assume that the weir discharge is correct, then these curves show the percentage difference between a rating in still water and a rating in moving water, and furnish the data for preparing a rating table for moving water from the one for still water. We have only to apply the correction to any velocity for any given number of revolutions per second, shown on the proper diagram, to obtain the velocity in moving water. This has been done, and the rating tables of the meters in still and in moving water are given in the following table. The velocity in moving water for any given number of revolutions per second is greater for all speeds of the Price meters than the velocity in still water. For the Haskell meter (a propeller instrument) the velocity is less in moving water than in still water for low velocities, and probably about the same in moving water as in still water for the higher velocities.[a] The discharges might now be computed, using the rating tables for moving water, and they would agree closely with the weir discharges. New vertical velocity curves have been found for series A, and the mean velocity ascertained from them. The shape of the new curve differs little from the old one, as the range of velocities for any curve is small. The new velocities obtained from the corrected curves are given in the second column of the table.

Rating tables of current meters in still water and in moving water in Cornell University experiment canal.

Revolutions per second.	Small Price No. 351.[b]		Large Price No. 88.[b]		Haskell No. 3.[c]		Fteley.[b]		Revolutions per second.	Haskell No. 3.[c]		Fteley.[b]	
	V	V_w	V	V_w	V	V_w	V	V_w		V	V_w	V	V_w
									1.10	1.30	1.28	1.18	1.07
									1.20	1.40	1.39	1.29	1.15
0.10	0.284	0.318							1.30	1.50	1.49	1.39	1.23
0.15	0.397	0.437	0.54	0.660					1.40	1.60	1.60	1.50	1.30
0.20	0.510	0.552	0.70	0.804					1.50	1.71	1.70	1.61	1.38
0.25	0.626	0.664	0.86	0.950					1.60	1.81	1.81	1.72	1.45
0.30	0.741	0.776	1.03	1.097					1.70	1.92	1.92	1.82	1.53
0.35	0.857	0.887	1.19	1.244					1.80	2.02	2.03	1.93	1.60
0.40	0.972	0.998	1.35	1.392					1.90	2.13	2.14	2.04	1.67
0.45	1.088	1.109	1.52	1.550					2.00	2.23	2.24	2.14	1.74
0.50	1.204	1.220	1.68	1.709	0.70	0.647	0.536	0.552					
0.55	1.319	1.332	1.84	1.874	0.75	0.698	0.589	0.598					
0.60	1.435	1.447	2.01	2.042	0.80	0.751	0.643	0.644					
0.65	1.550	1.562	2.17	2.207	0.85	0.803	0.697	0.690					
0.70	1.660	1.677	2.34	2.373	0.90	0.855	0.750	0.735					
0.75	1.782	1.796	2.50	2.538	0.95	0.907	0.804	0.779					
0.80	1.897	1.915	2.66	2.702	1.00	0.960	0.858	0.822					
0.85	2.013	2.033	2.83	2.868	1.05	1.014	0.911	0.864					
0.90	2.129	2.150	2.99	3.034	1.10	1.068	0.965	0.905					
0.95	2.244	2.266	3.15	3.199	1.15	1.121	1.018	0.946					
1.00	2.36	2.38	3.32	3.366	1.20	1.174	1.072	0.987					

[a] Recent experiments with a Fteley meter do not verify this statement.
[b] Meter held rigidly with rod.
[c] Meter held with cable.

NOTE.—V=velocity, in feet per second, from still-water rating, V_w=velocity, in feet per second, from moving-water rating.

A reason for the percentage increase in the variation of the meter discharge from the weir discharge as the velocity decreases is to be found in the rating table from which the velocities are computed. The experiment of November 1 shows that the small Price meter will measure a velocity of 0.23 foot per second, as the mean velocity found from the weir is 0.229 foot per second and the meter wheel revolved with regularity at all points where it was held. The observations in the rating do not extend to this velocity, however, and do not warrant the preparation of a rating table giving velocities of less than 0.5 foot per second.

In the three ratings of small Price meter No. 351 on May 9, 1901, shown in fig. 29, page 89, there are only four observations for velocities less than 0.5 foot per second and none below 0.45 foot per second. The extension of the table to velocities less than 0.5 foot per second is probably made on the assumption that this part of the rating curve is a straight line. That this is very far from the truth can be seen from an inspection of the three curves of fig. 29. Another reason is that discharge shown by the Cornell University standard weir is probably from 1 to 3 per cent too large for low heads on the weir. Recent experiments seem to indicate this fact. These two causes, however, account for only a comparatively small part of the difference between current-meter and weir discharge at low velocities which the experiments of series A and C, and others made since the writing of the body of this paper, indicate. The failure of the meter to indicate as high a velocity when held in a very slow-moving current, as when the meter is dragged through still water, is probably due to the stream lines becoming more and more oblique to the axis of the canal as the velocity decreases, and consequently their impact on the meter being less than when they moved more nearly in the direction of the axis of the canal.

ERROR IN USING AVERAGE OF BOTTOM AND SURFACE VELOCITIES FOR MEAN VELOCITY.

The following table gives the mean velocity and the average of the velocities 0.5 foot above the bottom and 0.5 foot below the surface for the experiments of series A, C, and D; also the error, in per cent, made by using the average of surface and bottom velocities instead of the mean velocity. It is seen that in all these experiments except two the mean velocity is greater than the half sum of the surface and bottom velocities, the difference varying from −2.2 per cent to +30.6 per cent.

Table showing relation between mean velocity and average of surface and bottom velocities as found from Cornell University standard weir and from current meters in Cornell experiment canal.

No. of experiment.	Series A.			Series C.			Series D.		
	V	V'	$\frac{V-V'}{V}$	V	V'	$\frac{V-V'}{V}$	V	V'	$\frac{V-V'}{V}$
	Feet per second.	*Feet per second.*	*Per cent.*	*Feet per second.*	*Feet per second.*	*Per cent.*	*Feet per second.*	*Feet per second.*	*Per cent.*
1	1.164	0.987	+15.3	1.409	1.412	− 0.2	1.528	1.436	+6.2
2	0.573	0.416	+27.5	0.932	0.954	− 0.2	1.528	1.436	+6.2
3	.....	.....	.....	0.477	0.468	+ 0.2	.....	.....	.....
4	1.268	1.190	+ 6.1	0.265	0.231	+12.8	.....	.....	.....
5	0.841	0.749	+10.9	1.444	1.364	+ 5.6	.....	.....	.....
6	1.462	1.384	+ 5.4	1.018	0.940	+ 7.7	.....	.....	.....
7	1.056	0.965	+ 8.6	0.502	.....	.....	.....	.....	.....
8	1.056	0.946	+10.4	0.255	0.177	+30.6	1.905	1.807	+5.2
9	.....	.....	.....	0.515	0.435	+15.5	1.905	1.779	+6.6
10	.....	.....	.....	1.914	1.730	+ 9.6	1.649	1.613	+2.2
11	0.980	0.887	+ 9.5	1.283	1.129	+12.0	1.649	1.600	+3.6
12	0.643	0.476	+25.9	0.893	0.759	+15.0	0.955	0.973	−1.9
13	0.788	0.687	+12.7	0.478	0.422	+11.8	0.955	0.897	+6.1
14	.....	.....	.....	1.454	1.311	+ 9.8	.....	.....	.....
15	.....	.....	.....	0.915	0.784	+14.3	0.689	0.704	−2.2
16	.....	.....	.....	0.229	0.179	+21.8	0.689	0.659	+4.3

V = mean velocity found from standard weir.
V' = mean of velocities 0.5 foot above bottom and 0.5 foot below surface, as found by meters uncorrected.

From the foregoing table, and from the table on page 79, in which is given the ratio of bottom velocity to mean velocity, it appears that bottom velocity is not a desirable quantity to use in computing discharge, for it varies between too wide limits. In a stream with a stony or gravelly bottom the limits must be considerably greater than in the Cornell canal with its smooth bottom. Observations at mid depth or at six-tenths depth give better results.

COMPARISON OF RESULTS BY THREE METHODS OF OPERATING METER.

Three methods of operating meters have been used in these experiments, viz, the ordinary method, the six-tenths-depth, and the integration. A detailed account of the manner of operating the meter by each of these methods has been given on pages 63 to 64. The time required to make a discharge measurement was from thirty to sixty minutes with the ordinary method, from twenty to thirty minutes with the six-tenths-depth, and from five to ten minutes with the integration.

The results by the ordinary method in series A to D (fig. 16, p. 69) give for each meter a well-defined line, which agrees with the weir line (0–0′) within about 2 per cent for velocities greater than 1.5 feet per second, the meter discharge being less than the corresponding weir discharge.

The agreement with the weir discharge is closer in series E than in series A to D, being −0.03 per cent for the Haskell meter, −1.13 per cent for Price meter No. 363, and −1.88 per cent for Price meter No. 351. The results by the six-tenths-depth method do not give as well-

defined a line (fig. 17) for each meter as do those by the ordinary method, the meter discharge being from 1 to 6 per cent greater than the corresponding weir discharge, and the mean being 3.5 per cent for small Price meter No. 363. If the observations with the latter meter had been taken at sixty-four-hundredths depth instead of at six-tenths depth the line representing the mean would coincide with the weir line 0–0′, and the greatest percentage variations from the mean would be −2.5 per cent and +2.5 per cent.

The results by the integration method do not give a well-defined line. The meter discharge is from 1 per cent to 9 per cent greater than the weir discharge for velocities greater than 0.5 foot per second in the experiments of series C and D, and from 0 to 5 per cent less than the weir discharge for the experiments of series A. The following table gives the true velocity as found from the weir and the velocity found by the integration method for different speeds of the meter; also the ratio of difference between these velocities and the true velocity, expressed in per cent:

Table showing effect of variation of speed of meter in integration method.

No. of experiment.	Date.	Depth.	True velocity by weir (V) per second.	Speed of meter per second.	Velocity of water by meter (V_1) per second.	$\frac{V-V_1}{V}$	Kind of meter.
	1900.	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Per cent.*	(a)
1	May 8	8.10	1.268	0.101	1.268	0.00	S. P. 351.
2	.....do	8.10	1.268	0.152	1.235	+ 2.60	Do.
3	.....do	8.10	1.268	0.270	1.200	+ 5.35	Do.
4	.....do	7.54	0.841	0.079	0.834	+ 0.09	Fteley.
5	.....do	7.54	0.841	0.055	0.842	− 0.01	Do.
6	May 9	7.85	1.056	0.056	1.028	+ 2.65	S. P. 351.
7	.....do	7.85	1.056	0.043	1.008	+ 4.54	L. P. 88.
8	May 15	7.73	0.980	0.032	0.978	+ 0.02	S. P. 351.
9	October 24	9.46	1.409	0.118	1.429	− 1.42	S. P. 363.
10	.....do	9.37	0.932	0.142	0.982	− 5.36	Do.
11	.....do	9.34	0.477	0.164	0.485	− 1.68	Do.
12	.....do	8.92	0.265	0.107	0.308	−16.23	Do.
13	October 26	7.67	1.444	0.137	1.519	− 5.20	Do.
14	.....do	7.64	1.018	0.135	1.079	− 5.99	Do.
15	October 27	7.58	0.502	0.143	0.522	− 3.99	Do.
16	.....do	7.56	0.255	0.128	0.312	−22.35	Do.
17	.....do	6.01	0.515	0.142	0.555	− 7.77	Do.
18	.....do	6.28	1.940	0.180	2.030	− 4.64	Do.
19	.....do	6.40	1.283	0.154	1.338	− 4.29	Do.
20	.....do	6.32	0.893	0.188	0.920	− 3.02	Do.
21	October 29	8.48	0.478	0.151	0.492	− 2.93	Do.
22	October 31	8.47	1.454	0.293	1.552	− 6.74	Do.
23	.....do	8.51	0.915	0.258	1.002	− 9.51	Do.
24	November 1	8.46	0.229	0.185	0.319	−39.30	Do.
25	December 6	9.51	1.528	0.253	1.540	− 0.78	Do.
26	.....do	9.51	1.528	0.284	1.572	− 2.87	Hask. 3.
27	December 7	9.32	1.535	0.243	1.576	− 2.66	Do.
28	December 8	7.25	1.905	0.252	1.962	− 2.99	Do.
29	.....do	7.22	1.649	0.295	1.748	− 6.00	Do.
30	.....do	7.18	0.955	0.254	1.039	− 8.79	Do.
31	.....do	5.15	0.689	0.179	0.752	− 9.14	Do.

a S. P.=small Price; L. P.=large Price; Hask.=Haskell. Figures given are the numbers of the meters.

Table showing effect of speed of meter on observed velocity as found by the integration method in experiments of table on page 84.

Experiments Nos. 9 to 25, omitting Nos. 12, 16, and 24.						Experiments Nos. 1 to 8.			
Speed of meter 0.10 to 0.14.	$\frac{V-V_1}{V}$	Speed of meter 0.14 to 0.18.	$\frac{V-V_1}{V}$	Speed of meter 0.18 to 0.30.	$\frac{V-V_1}{V}$	Speed of meter 0.05 to 0.15.	$\frac{V-V_1}{V}$	Speed of meter 0.15 to 0.30.	$\frac{V-V_1}{V}$
Feet per second.	*Per cent.*	*Feet per second.*	*Per cent.*	*Feet per second.*	*Per cent.*	*Feet per second.*	*Per cent.*	*Feet per second.*	*Per cent.*
0.118	−1.42	0.142	−5.36	0.180	−4.64	0.056	+2.65	0.152	+2.60
0.137	−5.20	0.164	−1.68	0.188	−3.02	0.043	+4.54	0.270	+5.35
0.135	−5.99	0.143	−3.99	0.293	−6.74	0.032	+0.02	.....	.....
0.130	−4.20	0.142	−7.77	0.258	−9.51	0.101	0.00	.....	.....
.....	.....	0.154	−4.29	0.253	−0.78	0.079	+0.09	.....	.....
.....	.....	0.151	−2.93	0.234	−4.94	0.055	−0.01	.....	.....
.....	.....	0.159	−6.33	.....	.....	0.061	+1.22	0.211	+3.97

This ratio is positive for all the experiments of series A, and is negative for those of C and D. This difference of sign is due to two causes: (1) The meter was moved faster in the experiments of C and D than in those of series A; and (2) it was moved through only 14 feet of the width in series C and D and through nearly 15.5 feet of the width in series A, and the additional 1.5 feet of width had a small velocity. It is seen from the above table that for low velocities the error in velocity increases as the speed of the meter increases. The table on page 84 shows this effect for velocities from 0.5 foot to 2 feet per second

COMPARISON OF RESULTS OBTAINED IN 1900 WITH THOSE OBTAINED IN 1901.

Examining the plotted results in figs. 16 (p. 69) and 18 (p. 70), it is seen that the meter discharge, compared with the corresponding weir discharge, is somewhat larger for the work of 1901 than for that of 1900 for velocities greater than 1.4 feet per second. This can not be due to change in rating of the meters, for an increase in the friction in a meter would reduce the observed velocity and make the meter discharge too small. There was considerably more seepage into the canal when the experiments of series E (1901) were made than at the time those of A, B, C, and D (1900) were made. There was also quite a little surface water entering the canal between the weir and the meter section in the experiments of series E. It is hardly possible, however, that this inflow between the weir and the meter section could amount to 2 per cent of the canal discharge. It must be borne in mind in this connection that the discharges obtained in 1900 for velocities greater than 1.4 feet were in shallow water, and therefore are not so reliable as those obtained in series E.

EFFECT OF METHOD OF HOLDING METER IN WATER.

In some of the experiments the meter was suspended from a cable; in others it was held in the water with a rigid rod. In the latter cases

its axis was free to move about a horizontal axis; in the former cases it was free to move about a vertical as well as a horizontal axis. When used with a cable a sinker is necessary, which, being attached to the under side of the meter, keeps the axis of the revolving wheel farther from the bottom than is desirable in some cases. It is also difficult to keep the meter in any desired place when held with a cable, and it usually changes its position considerably in a swift current. It was in order to measure the velocity close to the bottom and keep the meter at a desired place during the observation that in many of the experiments the meter was held with a rod. In ordinary river gaging, however, the Price and Haskell meters are held with cables and the meters are rated in still water suspended from a car and cable. The question then arises: "Is the rating table for a meter used suspended from a cable applicable to the same meter held with a rigid rod?"

On December 30, 1900, Mr. E. G. Paul made two ratings of small Price meter No. 351 in still water, one with the meter held with a rigid rod and the other with the meter held with a cable and a 5-pound lead sinker. In the following tables (pp. 87 and 88) are given the observations and computations of these ratings. The second column ("time in seconds") gives the average time required for the car to which the meter was attached to move over a run of 100 feet, and again over the same course in the opposite direction; the third column gives the number of revolutions observed in this average time; the seventh column gives for each of these cases the difference between the observed velocity and the velocity computed from the old rating table of April 23, 1900, for the same meter held with a cable. The tenth column of the first table (p. 87) gives the difference between the observed velocities and those computed from the new rating table, which is based on meter rated in still water and held by a rigid rod. The portion of this new rating table, which was used in reducing the Cornell experiments, is given on page 81. By comparing the corresponding quantities in the seventh and tenth columns of this table it will be seen how much more closely the values in the new rating table fit the observations than do those of the old one. The tenth column shows also how much the relation between the revolutions per second of this meter and the velocity as shown by these observations differs from the "most probable" straight line, $y=0.047+2.313x$, y being velocity in feet per second and x revolutions of meter wheel per second. From the seventh column of the table on page 87 it is seen that the observed velocities are larger than the computed velocities, indicating more friction in the meter on December 30, 1900, than on April 23, 1900.

The "most probable" linear relation between x and y for this case of the meter held with a cable was found to be $y=0.067+2.321x$. The differences in the tenth column show that this line does not fit the

observations. It crosses the curve given by the observations twice as y increases from 1 to 8 feet per second. The rating table was prepared by using this line as a basis, applying corrections as indicated by the corresponding difference of the tenth column, so that the rating table will difference smoothly. The numbers in column 12 show how closely the observed velocities agree with those computed from the new rating table with the meter held with a cable.

It should be noted that the smallest observed velocity is about 0.5 foot per second in one case and 0.35 foot in the other case.

Observations and computations of a rating of small Price meter No. 351 held by rigid rod, made at Chevy Chase, Md., December 30, 1900.

1.	2 and 3.		4.	5.	6.	7.	8.	9.	10.	11.
No. of observation.	Length of run, 100 feet.		Revolutions per second (x).	Observed velocity (y).	Computed velocity (y^1).[a]	Difference ($y - y^1$).	x^2	xy	Residual (v).	v^2
	Time.	Revolutions of meter.								
	Seconds.			*Feet per second.*	*Feet per second.*	*Ft. per second.*				
1	9.25	43.00	4.648	10.811	10.815	−.004	21.604	50.245	+.059	.003481
2	10.5	42.50	[b]4.048	[b]9.524	9.405	+.119				
3	16.0	43.00	2.688	6.250	6.231	+.019	7.225	16.800	−.014	.000196
4	22.5	43.00	1.911	4.444	4.414	+.030	3.652	8.493	−.022	.000484
5	35.0	42.25	1.207	2.857	2.797	+.060	1.457	3.448	+.018	.000324
6	45.0	42.00	0.933	2.222	2.183	+.039	0.870	2.073	+.017	.000289
7	53.0	42.00	0.794	1.887	1.867	+.020	0.630	1.498	+.003	.000009
8	54.5	42.00	0.774	1.835	1.823	+.012	0.599	1.420	−.003	.000009
9	68.0	41.75	0.614	1.471	1.461	+.010	0.377	0.903	+.004	.000016
10	85.5	41.75	0.488	1.170	1.174	−.004	0.238	0.571	−.006	.000036
11	106.5	41.00	0.385	0.939	0.944	−.005	0.148	0.362	+.001	.000001
12	118.0	41.00	0.363	0.885	0.891	−.006	0.132	0.321	−.002	.000004
13	154.0	40.50	0.263	0.649	0.669	−.020	0.069	0.171	−.006	.000036
14	197.0	39.50	0.201	0.508	0.522	−.014	0.040	0.102	−.004	.000016
15	220.0	39.00	0.177	0.455	0.569	−.014	0.031	0.081	−.001	.000001
16	293.0	36.90	0.126	0.341	0.351	−.010	0.016	0.043	−.003	.000009
			15.572	36.724			37.068	86.531		.004911

[a] Computed from rating table prepared from rating of April 23, 1900.
[b] Omitted in computing Σx and Σy.

NOTE.—Observation equation, $y-a-bx=v$; normal equations, $a\Sigma x+b\Sigma x^2=\Sigma xy$, $na+b\Sigma x=\Sigma y$. For these observations these become $15.572\ a+37.088\ b=86.531$, $15a+15.572b=36.724$. The solution of these gives $b=2.313$ and $a=.047$. The mean error of a single observation is $\sqrt{\frac{\Sigma v^2}{n-1}}=.019$. The probable error of result is $6.745\sqrt{\frac{\Sigma v^2}{n(n-1)}}=.003$.

Observations and computations of a rating of small Price meter No. 351 held by cable, made at Chevy Chase, Md., December 30, 1900.

1.	2 and 3.		4.	5.	6.	7.	8.	9.	10.	11.	12.
No. of observation.	Length of run, 100 feet.		Revolutions per second (x).	Observed velocity (y).	Computed velocity (y').[a]	Difference ($y-y'$).	x^2	xy	Residual. (r).	Computed velocity (y'').[b]	Difference ($y-y''$)
	Time.	Revolutions of meter.									
	Seconds.			*Ft. per second.*	*Ft. per second.*	*Ft. per second.*				*Ft. per second.*	
1	10.5	42.00	4.000	9.524	9.300	+.224					
2	12.5	42.50	3.400	8.000	7.900	+.100	11.560	27.200	+.042	7.98	+.02
3	16.5	42.50	2.576	6.061	5.967	+.094	6.636	15.613	+.015	6.04	+.021
4	21.5	42.75	1.990	4.651	4.588	+.063	3.960	9.256	-.034	4.68	-.029
5	26.0	42.50	1.635	3.846	3.767	+.079	2.673	6.288	--.015	3.857	+.009
6	29.0	42.50	1.465	3.448	3.383	+.065	2.146	5.051	-.020	3.443	-.005
7	31.0	42.25	1.363	3.226	3.151	+.075	1.858	4.397	-.004	3.290	+.017
8	41.0	42.50	1.037	2.439	2.411	+.028	1.075	2.529	-.084	2.451	.012
9	50.0	42.50	0.850	2.000	1.990	+.010	0.723	1.700	-.089	2.024	.024
10	61.0	42.00	0.688	1.639	1.624	+.015	0.473	1.128	-.024	1.655	-.016
11	70.0	42.00	0.600	1.429	1.430	-.001	0.360	0.857	.030	1.454	-.025
12	80.5	41.25	0.512	1.242	1.226	+.016	0.262	0.636	-.013	1.256	-.014
13	90.5	40.00	0.442	1.105	1.072	+.033	0.195	0.488	+.012	1.098	+.007
14	105.5	39.00	0.370	0.949	0.908	+.041	0.137	0.351	+.024	0.938	+.011
15	125.0	38.00	0.304	0.800	0.759	+.041	0.092	0.243	+.028	0.790	+.010
16	140.0	37.00	0.264	0.714	0.671	+.043	0.070	0.189	+.035	0.697	+.017
17	171.0	36.00	0.215	0.585	0.556	+.029	0.046	0.126	+.019	0.585	-.000
18	184.0	36.00	0.196	0.543	0.507	+.036	0.038	0.106	+.022	0.541	+.002
19	211.0	35.75	0.169	0.474	0.452	+.022	0.029	0.080	+.015	0.479	-.005
			18.076	43.151			32.334	76.289			

[a] Computed from rating of April 23, 1900.
[b] Computed from rating of December 30, 1900.

NOTE.—Most probable straight line given by these observations, $y=.067+2.321x$.

The following table shows the percentage difference of velocity for each 0.05 revolution for these two ways of holding the meter. It is seen that a given number of revolutions per second of this meter indicates a higher velocity when it is held with a cable than when it is held with a rigid rod, and that this percentage difference decreases as the velocity increases. In other words, in a given velocity the meter will revolve faster when held with a rigid rod than when held with a cable. It must be remembered that the values given in this table for velocities less than about 0.5 foot per second are only approximate, since the observations do not extend below 0.35 foot per second. We may say that if the rating table for this meter held with a cable be used to reduce observations made with the meter held with a rigid rod, the results will be in error from 1.3 per cent to 8.5 per cent for velocities from 1.5 feet to 0.5 foot per second.

Table showing percentage difference in velocity as shown by small Price meter No. 351 when rated in still water on a rigid rod and on a cable.

Speed of meter.	V_r	V_c	$\frac{V_r - V_c}{V_r}$	Speed of meter.	V_r	V_c	$\frac{V_r - V_c}{V_r}$
R. per sec.	*F. per sec.*	*F. per sec.*	*Per cent.*	*R. per sec.*	*F. per sec.*	*F. per sec.*	*Per cent.*
0.15	0.397	0.433	9.1	0.60	1.435	1.455	1.4
0.20	0.510	0.550	7.8	0.65	1.550	1.568	1.2
0.25	0.625	0.666	6.6	0.70	1.666	1.682	1.0
0.30	0.741	0.781	5.4	0.75	1.782	1.796	0.8
0.35	0.857	0.894	4.3	0.80	1.897	1.910	0.7
0.40	0.972	1.006	3.5	0.85	2.013	2.024	0.5
0.45	1.088	1.118	2.8	0.90	2.129	2.138	0.4
0.50	1.204	1.230	2.2	0.95	2.244	2.252	0.4
0.55	1.319	1.342	1.7	1.00	2.36	2.37	0.3

V_r=velocity with meter rated on a rigid rod.
V_c=velocity with meter rated on a cable.

Small Price meter No. 351 was again rated three times on May 9, 1901. It was held (1) with a cable, using an 18-pound sinker; (2) with a rod, the meter being free to tip; and (3) with a rod and the meter not free to tip. Space will not permit the insertion of the observed data or the computations, but the results are plotted in fig. 29, using R= revolutions per second as abscissæ and $K=\frac{V}{R}$=ratio of velocity to revolutions as ordinates. It is seen that these rating observations give a separate curve for each of these cases. The value of K for any value of R for one curve is very different from that for the other curves for velocities less than about 3 feet per second. For example, for R=0.3, K=2.755 for case 1 (curve AE), 2.585 for case 2 (curve BE), and 2.485 for case 3 (curve DF). The coefficient is largest when the meter has the most freedom of motion and least when it has no freedom of motion. As the velocity for a given number of revolutions varies directly with K, the velocity decreases as the freedom to tip about a horizontal axis and to swing around a vertical axis decreases. Or, stated in the other way, in a current of a given velocity the revolutions increase as the freedom of motion decreases. Each of these cases therefore requires a separate rating table.

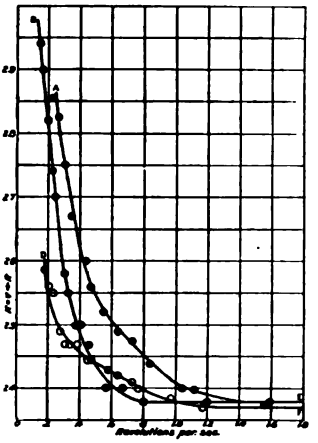

FIG. 29.—Curves showing relation between revolutions of meter and ratio between velocity of water and revolutions of meter from three ratings of small Price meter No. 351 at Chevy Chase, Md., May 9, 1901. Crossed circles are for observations with meter held with cable, an 18-pound sinker being used. Heavy black dots are for observations with meter held with rod and free to tip. Open circles are for observations with meter held with rod and not free to tip.

SURFACE VELOCITY EXPERIMENTS WITH FLOAT RODS AND WITH CURRENT METERS.

It was noticed during the weir and current-meter comparisons that the shape of the vertical velocity curve near the surface obtained with a small Price meter was quite different under certain conditions from that obtained with the Haskell meter under the same conditions. This peculiarity can be seen in the simultaneously obtained curves of fig. 30. The Haskell meter indicates a higher surface velocity than the Price meter. By surface velocity obtained with a meter is meant that obtained when the revolving part of the meter is entirely under water all of the time and yet as close to the surface as possible. For the three meters used in these surface velocity observations the distance of the

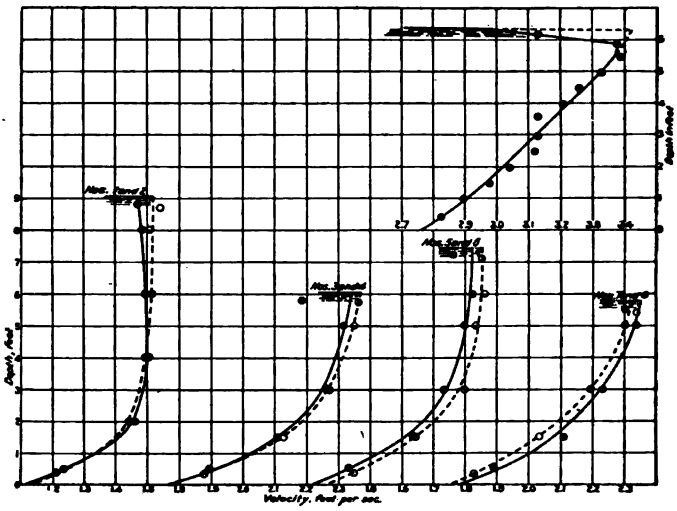

FIG. 30.—Vertical velocity curves from simultaneous experiments of series E with small Price meter No. 363 and with Haskell meter No. 3. Large black dots are for Price meter, open circles for Haskell meter.

center below the surface was as follows: Small Price meter No. 351, from 2.5 to 3 inches; Fteley No. 107, from 3 to 3.5 inches; Haskell No. 3, from 4 to 4.5 inches.

In order to investigate this matter further the surface velocity of a part of the canal was measured simultaneously with floats and with two current meters. The floats used were 6-inch cubes of wood loaded with lead so as to have an immersion of 5¼ inches. Surface floats of wood 6 inches by 6 inches by 1 inch thick were also used. These were started near the center of the canal 18 feet above the upper wire and timed over a run of 60 feet. Any that passed the lower wire within 2 feet of either side were not used in computing velocity. The meters were held 10 feet below the lower wire, the Haskell at 5 and 7 feet from the south side of the canal, the Price at 9 and 11 feet from the

south side, and the revolutions were counted for periods of forty seconds. The results of the experiments are given in the following table:

Comparison of surface velocity measured with float rods and with current meters.

Small Price meter No. 351.				Haskell meter No. 3.			
Number of float observations.	Velocity with floats.	Number of 40-second periods.	Velocity with meter.	Number of float observations.	Velocity with floats.	Number of 40-second periods.	Velocity with meter.
	Feet per second.		*Feet per second.*		*Feet per second.*		*Feet per second.*
11	3.122	10	2.794	13	3.083	13	3.171
12	3.076	12	2.806	12	3.144	12	3.187
13	3.030	13	2.818	10	3.135	10	3.165
36	3.076	35	2.806	35	3.121	35	3.174

During the foregoing observations the depth of water in canal was 6.04 feet, its mean velocity 2.8 feet, and the discharge 275.3 cubic feet per second. The float velocity that is compared with the Price meter velocity is obtained from the floats passing the lower wire from 2 to 8 feet from the south side of canal; that which is compared with the Haskell meter velocity is found from the floats passing the lower wire from 8 to 14 feet from the south side of canal. It is seen that in each experiment the float velocity is greater than that of the Price meter and less than that of the Haskell meter. The mean float velocity for the three experiments is 8.8 per cent greater than the velocity with the Price meter and 1.7 per cent less than that with the Haskell meter.

It was thought that possibly this failure of the small Price meter to give correct surface-velocity indications was due to wave motion of the meter. Another experiment was therefore made with the floats and with the meter held with a rod and not free to tip. Fteley electric meter No. 107 was also used in place of the Haskell meter. The floats were started at many points in the width of the canal and were timed over a run of 50 feet. Those that came within 2 feet of either bank were not used in computing the velocity. The meters were held 24 feet below the lower wire at points from 0 to 7 feet from the center, and the revolutions were counted for 50-second periods.

The mean surface velocity shown by the small Price meter from thirty-two 50-second periods is 1.967 feet, and the corresponding float velocity obtained from ten floats passing from 2 to 8 feet from the south side of canal is 2.146 feet. The corresponding surface velocity shown by the Fteley meter is 2.074 feet, and by the floats from 8 to 14 feet from the south side of canal, 2.137 feet. The float velocity is 8.3 per cent greater than the Price meter velocity and 2.95 per cent greater than the Fteley meter velocity. The difference between the

float and the Price meter velocities is about the same as in the previous experiments.

During this experiment the Price meter was held 3.5 feet from the south side of canal, with its center 2¾ inches below the surface for seven 50-second periods, and again with its center 4.5 inches below the surface for about the same length of time. The mean velocity at 2¾ inches depth was 1.884 feet, and at 4.5 inches depth it was 2.035 feet. The float velocity at this point was 2.137 feet. The indicated velocity increased 7.4 per cent as the meter was lowered 1¾ inches. The float work in this experiment is not very satisfactory.

On May 20 vertical-velocity-curve data were taken with small Price meter No. 351 and with Haskell meter No. 3 for different mean velocities, with a view to determining the magnitude of this error at different velocities and depths below the surface. The Haskell meter was held with a cable 1 foot south of the center of the canal and 244 feet from the weir; the Price meter was held with a cable 1 foot north of the center of canal. The following data were obtained:

Velocities near the surface of Cornell University experiment canal as found with small Price meter No. 351, and with Haskell meter No. 3.

Number of run.	Small Price meter No. 351.[a]				Haskell meter No. 3.[b]		
	Depth below surface, in inches.				Depth below surface, in inches.		
	2.5	4.5	6.5	8.5	4.5	6.5	8.5
Second	0.482	0.490	0.482	.....	0	0	0
Third	0.556	0.595	0.575	.....	0.592	0.615	0.602
Fourth	0.936	0.958	0.958	0.942	1.015	1.032	1.010
Fifth	1.297	1.281	1.320	1.299	1.348	1.325	.....
Sixth	1.480	1.457	1.389	1.434	1.450	1.392	1.412
Seventh	1.611	1.556	1.533	.....	1.552	1.572	.....

[a] Meter held with cable 1 foot north of canal center and 244 feet from weir.
[b] Meter held with cable 1 foot south of canal center and 244 feet from weir.

Each run lasted about half an hour, during which time the mean velocity remained constant. These results indicate that for velocities less than 1.5 feet the difference between the velocities indicated by these meters is small from the surface to a depth of 8.5 inches.

In the upper right-hand corner of fig. 30 is a vertical velocity curve prepared from thirty-five 50-second observations taken in forty minutes 7 feet from the south side of the canal, at a station 220 feet from the weir, with small Price meter No. 351. It shows a decrease of 8.5 per cent in the velocity 2¾ inches below the surface.

From these float and current-meter surface velocity experiments we conclude that a small Price meter will not measure velocity correctly when its center is within 0.5 foot of the surface, if the velocity be greater than 1.5 feet per second, and that this error increases from 0 at about 0.5 foot below the surface to 8 or 9 per cent at 2.5 inches below the surface.

LOW VELOCITY CURRENT METER AND WEIR COMPARISONS.

The current-meter and weir-discharge comparisons described on pages 58 to 80 seem to indicate that for velocities less than about 0.5 foot per second the discharge given by the meter is less than that given by the weir, and that this difference increases as the velocity decreases. With a view to examining this point further, small Price meter No. 375 was rated on October 26, 1901, and the Fteley meter No. 107 on November 1, in the 10-foot canal at Cornell University. A chronograph was used to indicate the time and revolutions. The rating in each case was carried to velocities less than 0.2 foot per second; for these lower velocities the motion of the car was not very steady. On October 25, November 1 and 20 comparative measurements of the discharge of the 10-foot canal were made with these two meters and with the standard weir, using the 0.6 depth method. The results are given in the following table:

Low velocity current meter and standard weir discharge comparisons.

Date.	Small Price meter No. 375.				Date.	Fteley meter No. 107.			
	Depth water.	Mean velocity.		$\frac{V_w - V_m}{V_w}$		Depth.	Mean velocity.		$\frac{V_w - V_m}{V_w}$
		Weir V_w	Meter V_m				V_w	V_m	
1901.	*Feet.*	*Feet.*	*Feet.*	*Per ct.*	1901.	*Feet.*	*Feet.*	*Feet.*	*Per ct.*
October 25	7.79	.564	.468	+17.0	November 1	8.79	1.257	1.277	− 1.6
Do	7.52	.401	a.166	+58.8	Do	8.58	1.098	1.129	− 2.8
Do	7.70	.508	a.390	+23.2	Do	8.86	.951	.957	− 0.7
Do	7.90	.630	.527	+14.7	Do	8.24	.872	.907	− 4.0
Do	8.02	.712	.646	+ 9.1	November 20	7.07	.154	.127	+17.5
Do	8.12	.776	.715	+ 7.9	Do	7.45	.361	.322	+10.8
Do	8.25	.863	.826	+ 4.3	Do	7.72	.529	.522	+ 1.3
Do	7.95	.666	.611	+ 8.9	Do	7.95	.674	.693	+ 2.9
November 1	9.05	1.454	1.427	+ 1.8	Do	7.31	.286	.253	+11.5
Do	8.79	1.257	1.259	+ 0.0	Do	7.08	.169	.142	+16.0
Do	8.58	1.098	1.080	+ 1.7					
Do	8.36	.951	.890	+ 6.4					
Do	8.24	.872	.830	+ 5.1					

a No measurable velocity in some parts of section.

The results for the Fteley meter are plotted in fig. 31, and those for the Price meter in fig. 32.

Some of these discharge measurements were made just previous to the rating of the meters, and the remainder only a short time after. They indicate, as in the other comparisons, that the meter discharge obtained in this canal by this method is less than the corresponding weir discharge for low velocities. Whether this difference was due to imperfect rating of the meter, to the method of measuring the discharge, to the condition of the water in this canal, or to the meters themselves was then to be considered. It was frequently noticed that the rate of rotation of the meter wheel was very irregular in this canal when the velocity was small.

The following table shows the irregularity of motion of the water in the 10-foot canal as compared with that in the 3-foot entrance to the 6-foot canal:

Table showing irregularity of motion of water in 10-foot canal and 3-foot entrance to 6-foot canal, Cornell University hydraulic laboratory.

Speed in 10-foot canal.				Speed of meter in 3-foot canal.			
Fteley meter.		Fteley meter.		Small Price meter.		Fteley meter.	
Revolutions.	Time.	Revolutions.	Time.	Revolutions.	Time.	Revolutions.	Time.
	Seconds.		*Seconds.*		*Seconds.*		*Seconds.*
5	32.0	5	20.9	3	24.0	8	27.0
5	23.4	5	27.0	3	25.0	8	27.0
5	22.9	5	22.7	3	24.3	8	27.2
5	25.1	5	25.0	3	24.0	8	27.3
5	28.5	5	24.9	3	24.7	8	27.5
5	23.9	5	24.9	3	24.1	8	26.8
5	23.0	5	26.9	3	24.6	8	28.0
5	29.0	5	26.1	3	24.4	8	27.9
5	28.3	5	24.3	3	24.0	8	27.0
5	28.7	5	23.7			8	27.2
5	31.1	5	28.0		24.34		
5	48.5	5	22.5				27.19
5	25.5	5	21.5				v=0.327
5	34.4	5	21.6				
5	27.0	5	25.9				
5	32.5	5	25.3				
	28.99		24.45				
	v=0.230		v=0.258				

It is seen that for a velocity of 0.23 foot per second in the large canal, the variation is from 21 per cent below to 67.3 per cent above the mean, and for a velocity of 0.253 foot per second the variation from the mean is from 14.5 per cent below to 14.5 per cent above the mean. In the entrance to the small canal the variation is only from 1.5 per cent below to 2.9 per cent above the mean for a mean velocity of 0.327 foot.

If this irregularity of motion of the water is due to conflicting currents which strike the current meters obliquely, the difference between meter and weir discharge may be mainly due to this cause. There is some reason, however, for thinking it may, in part at least, be due to the imperfect rating in still water. This Fteley meter No. 107 is one of the two meters used in the tests of low-water stream measurements described on pages 100 to 105. During the progress of this work it was rated at Chevy Chase, Md., by Mr. Stewart and the writer. This rating shows a change of about 1 per cent in velocities from 0.6 to 1.4 feet per second, and a much larger percentage change for very low velocities. This increase in friction of the meter shown by the two ratings was not anticipated. There was an interval of fourteen months between the ratings, but the meter had not been used much, and only in the laboratory. The conditions during the second rating were quite different from those of the first, and, on the whole, not so favorable for accurate work. In the first rating the water was clear, still, and warm. A chronograph being used, time and revolutions could be read

to tenths. The motion of the car was somewhat unsteady. In the second rating the water was muddy, cold, and had a small irregular velocity, and there was some wind. The motion of the car was steadier than during the first rating. The distance passed over by the car for whole number of revolutions was read to 0.5 foot. The rating table obtained from the second rating was used in computing the meter discharge of 4 feet wooden flume in test of low-water stream measurements, and these results show a close agreement of meter and

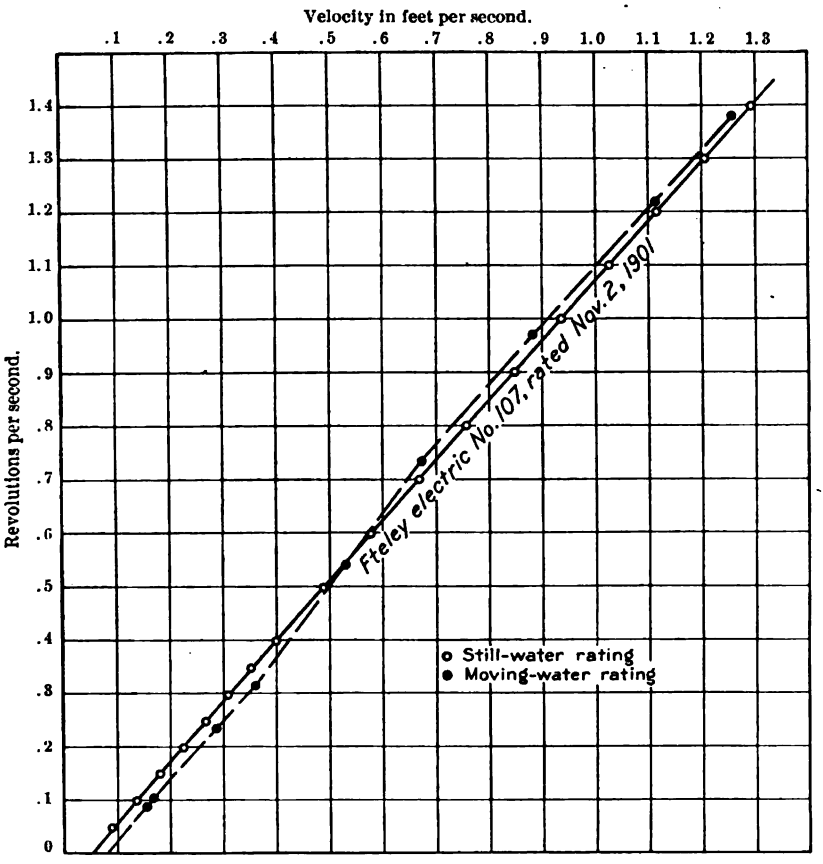
Velocity in feet per second.
.1 .2 .3 .4 .5 .6 .7 .8 .9 1.0 1.1 1.2 1.3
Revolutions per second.
1.4 1.3 1.2 1.1 1.0 .9 .8 .7 .6 .5 .4 .3 .2 .1 0
Fteley electric No. 107, rated Nov. 2, 1901
o Still-water rating
• Moving-water rating

FIG. 31.—Diagram showing results of rating in still and moving water, Fteley meter.

Cippoletti weir discharges even for low velocities. The first rating, on the face of it alone, is more accurate than the second; and as it is not uncommon for a meter to change its rating that amount in that time, the evidence is not conclusive.

In view of the fact that current meter and weir discharge measurements differ so widely for low velocities, whether due to the meter or to the condition of the water, the discharge should not be measured with a current meter in a section where the mean velocity in any considerable part of it is less than half a foot per second.

CURRENT METER AND PITOT TUBE COMPARISONS.

With a view to obtaining some data bearing on the adaptability of a Pitot tube for stream measurements, the writer obtained a tube of the Darcy-Ritter type, somewhat like that in fig. 2, and measured with it the discharge of the 6-foot canal in the Cornell University hydraulic laboratory. He also made some relative ratings of a Pitot tube of the point and wall pressure type, and two current meters in the entrance to this 6-foot canal. The velocity in this canal was small, and the tube was not a good one, or, at least, was not in good condition. The results, therefore, are not given. They serve only to

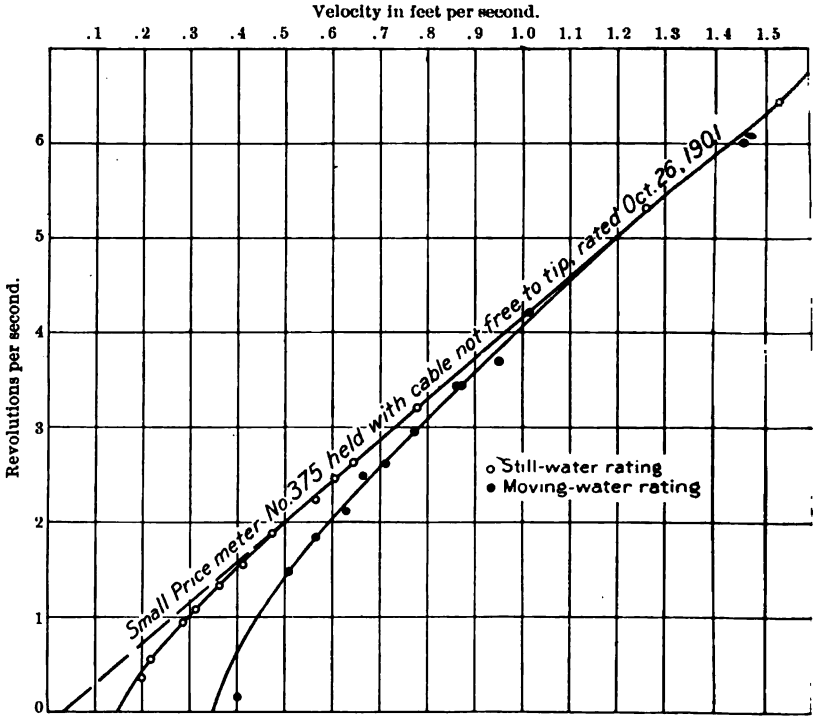

FIG. 32.—Diagram showing results of rating in still and moving water, small Price meter.

emphasize the fact, noted by others, that this instrument is poorly adapted to measurement of flow in open channels, and is vastly inferior to the current meter for this purpose.

APPLICATION OF RESULTS TO ORDINARY RIVER GAGING.

It has been shown that the discharge of the Cornell University canal can be measured with a small Price current meter by the ordinary point method with an error of not more than 1 per cent under favorable conditions. This degree of accuracy, however, can seldom be attained in ordinary river gaging, as the conditions are not so

favorable for accurate work. The bed of the Cornell canal is hard, smooth, and regular in shape; the bed of a river is frequently of soft material, into which the meter will settle, or is gravelly, stony, and irregular in shape. There are no shallow parts in the canal the depth and velocity of which it is difficult to measure with accuracy, but frequently there are one or more of these in a river discharge section. There is a good measurable velocity in all parts of the canal for all depths, while in a river there are frequently parts in which the velocity is too small for accurate measurement. The change in velocity from one point to another in a discharge section is frequently greater in a river than in the canal, on account of obstructions. Fluctuations of the surface elevation are also larger in a river than in the canal. On these accounts it is to be expected that ordinary river discharge measurements may be several per cent less accurate than those of this canal.

In river gaging work, however, it is not alone accurate discharge measurements that are required, but it is the accurate measurement of the volume passing the gaging station each day and each month. The accuracy of daily and monthly flow of a stream depends on the accuracy of the discharge curve, or the relation between the total discharge and the river stage, and on the accuracy of measurement of the fluctuation of river stage. The latter is quite as important as the former, and improvement in it should keep pace with improvements in the former.

ACCURACY OF RESULTS.

The discharge of the Cornell University standard weir is computed from Bazin's formula. It is not possible, however, to measure the head on the weir in exactly the same way that Bazin measured the head on his weir, so that this formula is not strictly applicable to this weir. From experiments made at the Cornell hydraulic laboratory, however, Prof. G. S. Williams believes that the actual discharge of the Cornell weir will not differ from that computed by Bazin's formula by more than 1 per cent for heads up to 1.5 feet, nor more than 3 per cent for heads up to 2.5 feet.

The results given in the tables have been obtained with five current meters operated in three ways by several observers. In several of the experiments simultaneous measurements were made with two meters, so that we have several checks on the work. We have not carried any of the computations beyond the third decimal place, believing that the length of a meter observation, the method of obtaining fractional parts of a revolution of the meter wheel, the uncertainty in the exact location of the vertical velocity curve, and the pulsations in the water do not warrant even this degree of accuracy. In closely studying the results some little inaccuracies will be found, as all the results

are not obtained in exactly the same way, though they are believed to be substantially correct.

In comparing the results of these experiments with results obtained by other experimenters it must be remembered that we have used smaller velocities in many of our experiments than were used by them. In Fteley and Stearns's comparisons (p. 58) the smallest mean velocity is 1.7 feet, in Henry's (p. 50) the smallest is 3 feet, in Bazin's (p. 43) the smallest is 1.25 feet, in Francis's (p. 54) it is 0.5 foot, and in Marr's (p. 49) 2.4 feet. For these and higher velocities our meter results by the ordinary method agree closely with those given by the weir. We are unable to find any comparisons by other experimenters for velocities less than 1 foot per second, except a few by Francis with rods and those at Cornell with rods (p. 57.) The large disagreement between meter and weir is for the low velocities, a field into which apparently few experimenters have entered.

CONCLUSIONS.[a]

The conclusions to be drawn from these experiments may be briefly summarized as follows:

(1) Discharge measured with current meter by the ordinary (point) method agrees with that given by the Cornell standard weir within 2 per cent for velocities above 1.5 feet per second. (See tables on pages 65 to 68 and 71.)

(2). For velocities less than 1.5 feet per second the discharge found with the Price meters is less than the corresponding weir discharge, and the difference increases rapidly as the velocity decreases. (See tables on pages 65 to 68.)

(3) For velocities less than about 1.5 feet per second the discharge found with the Haskell meter is greater than that of the weir, and the difference increases as the velocity decreases. This difference is 6 per cent for a velocity of 0.75 foot per second. (See fig. 16.)

(4) The discharge of the Cornell canal can be measured with a small Price meter, ordinary point method, with an error of not more than 1 per cent under favorable conditions, a velocity observation lasting fifty seconds being taken in each 2.3 square feet of discharge area.

(5) When the most accurate results are desired the meter should be held with a rod and not given freedom to tip.

(6) Velocities of 1.5 feet per second and upward obtained with a small Price meter when its center is closer to the surface than 0.5 foot are too small by from 0 to 9 per cent. This error, however, decreases from maximum at the surface to 0 at about 0.5 foot depth.

(7) The small Price meter will measure velocities of 1 foot per second and less more accurately than either the large Price meter or the

[a] See also pp. 137, 138, 150, 151, and 160.

Haskell meter. It appears from these experiments that the smallest velocity that these meters will measure with a fair degree of accuracy is 0.5 foot for the large Price meter and the Haskell meter and 0.22 foot for the small Price meter. (See fig. 16.)

(8) The small Price meter should be frequently rated, and it should be used with much care if accurate results are required.

(9) The six-tenths-depth method gives discharge from 2 to 6 per cent in excess of that by the weir, depending on the ratio of width to depth. (See fig. 17.)

(10) The integration method as a rule gives results in excess of those given by the weir, the difference increasing with the speed of the meter and decreasing as the velocity increases. (See fig. 17 and tables on pages 84 and 85.) By using special care in moving the meter at a slow speed and a uniform rate it may be possible to obtain better results by this method than those shown by these experiments. A device like Harlacher's (page 38) for giving the meter a uniform motion will increase the accuracy somewhat. We do not believe, however, that the little saving of time of this method over that by the mid-depth or the six-tenths-depth will warrant the use of any such device. The method is useful only as a rough check on one of the other methods.

(11) The thread of maximum velocity is at the surface for depths less than 2 feet and unobstructed flow at the lower end of the canal. For depths of 5 feet or more and discharge checked at the lower end of the canal this thread is from two-tenths to four-tenths depth below the surface, the mean for thirty-one experiments being thirty-one-hundreds depth. (See table on page 78.)

(12) The position of the thread of mean velocity varies from five-tenths depth for small depths to seventy-three-hundredths depth for the larger depths. For the thirty-one experiments by the ordinary method of series C and D it is sixty-four hundredths depth below the surface. (See table on page 78.)

(13) The surface velocity is always greater than the bottom velocity with center of meter 0.25 foot above bottom. (See figs. 23 to 28.)

(14) The ratio of bottom velocity to mean velocity varies from 0.6 to 0.9, being 0.8 for the experiments of series A, 0.75 for series C, and 0.85 for series D. (See table on page 79.)

(15) The ratio of mean velocity to mid-depth velocity varies from 0.90 to 0.99, the mean of forty experiments being 0.95. (See table on page 79.)

(16) The average of the velocities 0.5 foot above the bottom and 0.5 foot below the surface is from −2.2 per cent to +30 per cent less than the mean velocity shown by the weir. (See table on page 83.)

(17) A small Price meter will revolve faster in moving water of a given velocity when held with a rigid rod than when held with a

cable. (See table on page 89.) Hence the same rating table will not answer for both.

(18) The bottom velocity varies between so wide limits that it is not a desirable quantity to use in computing discharge; the mid-depth or the six-tenths-depth is better.

TESTS OF LOW-WATER STREAM MEASUREMENTS.

A knowledge of the summer flow of streams is very important, for on the minimum flow depends mainly the value of the stream for water-supply and power purposes. This summer flow is very small in some streams, in others it is nothing; even the larger ones become very shallow during the low water season. The importance of a knowledge of the smallest summer flow of streams has caused numerous measurements to be made of small amounts of water flowing, usually, in a broad shallow channel. On account of the expense of constructing a measuring weir on a stream the current meter is the instrument used for measuring minimum as well as ordinary flow. Frequently this mininum flow can not be measured at the gaging station on account of the small velocity. It is necessary to go up or down stream from the gaging station, and find a place where the depth is at least half a foot, bed not too rough, and the velocity from 0.5 to 1.5 feet per second. Some Western streams have a minimum flow of 0.75 to 4 second-feet, a gravelly or sandy bed, a minimum depth of 6 to 12 inches, and a velocity of 0.5 to 1½ feet per second. The velocity is computed from observation with a current meter held as near to the bed, and then as close to the surface as possible, (its center is usually from 0.15 to 0.20 feet from bed and surface). The mean velocity in a vertical is taken as the half sum of the bed and surface velocities.

In order to test the accuracy of current-meter work done under these conditions, the following series of comparisons of current-meter and weir discharges were made in the hydraulic laboratory of Cornell University during January and February 1903. They were planned to cover the conditions of quantity of water, depth, velocities and roughness of bed, that the writer met in his work in Nevada and in California.

Fig. 33 shows the 6-foot canal in the Fall Creek hydraulic laboratory of Cornell University. Water is admitted to it through two gates 14 feet upstream from the baffle B, and can be measured by the sharp crested weir F. In this canal was built, as shown, a rectangular wooden flume 4 feet wide, 2 feet deep, and 20 feet long, with smooth bottom and sides. C is a 4 by 4 inch timber which serves as a breakwater to destroy the energy of the falling water. D is a baffle made of 1 by 8 inch timber placed horizontally 2 inches center from center. G is a baffle like D but with strips 1 by 3 inches placed vertical to hold back the water and give the required depth for any discharge and per-

mit the water to pass out through all parts of the cross section. B is a bulkhead carrying a 2-foot Cippoletti measuring weir. The hook gage for showing the head on the weir is 5 feet upstream from the weir. The current-meter measurements were made at E, 4 feet from the

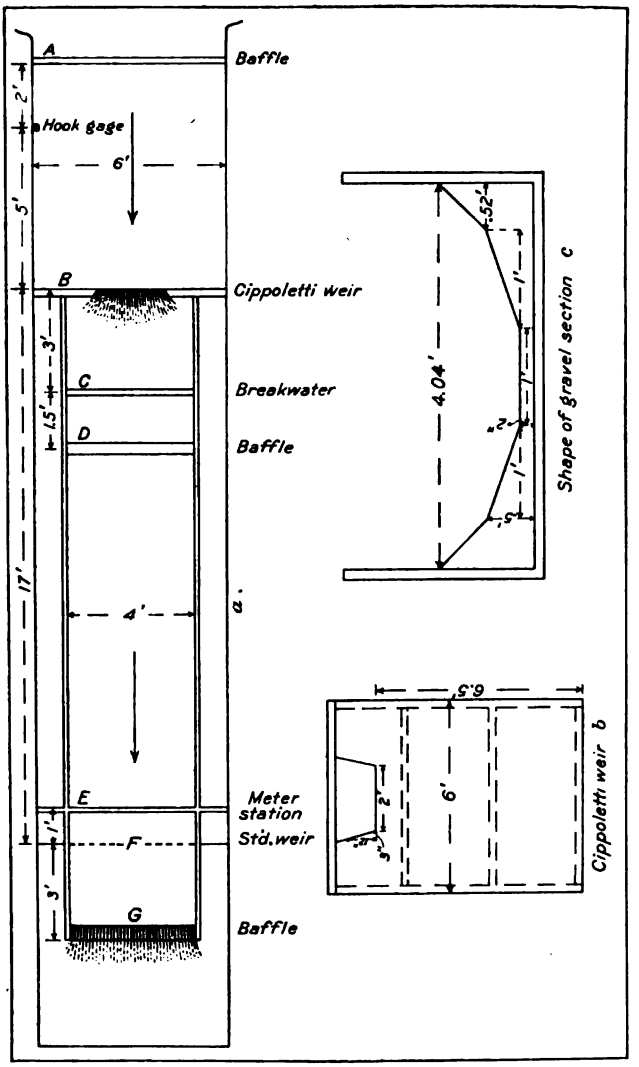

FIG. 33—Plan and sections of flume.

lower end of the flume. The meters were held with a rod clamped in proper positions to two 2 by 4 inch strips fastened to the sides of the canal.

Three series of discharge measurements were made with two meters. In the first series the bottom and sides of the flume were smooth, the wood

being planed hemlock 2 inches in thickness. In the second series the bed was mixed gravel, and had approximately the shape shown in fig. 33. Five of these wooden forms were fastened in the bottom of the flume 3 to 4 feet apart, and gravel placed between them. This gravel bed is shown in Pl. V. In the third series the bed was as in series 2, but with rounded stone 2 to 3½ inches in diameter scattered over the bed, these stones being the coarse material of pit gravel.

The current meters used were a small Price meter No. 396 (see Pl. I, p. 20), held with a rod and free to tip. The other was the eight-vaned Fteley meter (see fig. 5, p. 22) used in the experiments described on pp. 93–95.

In the first series the small Price meter was held with its center 0.15 foot from the bottom and 0.15 foot from the surface for periods of 50 to 100 seconds, and the mean velocity in a vertical was taken as the half sum of the surface and bottom velocities. The mean velocity in a vertical was found in the same way with the Fteley meter, but its center was held 0.2 foot from the bottom and 0.2 foot from the surface. The verticals in which velocity was measured were 0.5 foot apart, the end ones being 0.25 foot from the sides of the flume.

In the second series the velocity was measured in verticals from 0.5 to 1 foot apart. On January 28, 29, and 30 the meters were held as close to the bottom and surface as possible, viz, 0.15 foot in the case of the small Price and 0.2 foot in the case of the Fteley. On February 10, instead of holding the meters as close to the bottom as possible, they were each held with centers 0.3 foot above the bottom.

In series 3 both meters were held as near to the surface as possible, and with center 0.3 foot above bottom in each case.

The order of work was usually as follows: The gate was opened in order to admit about the quantity of water desired and the baffle *c* was adjusted so as to give the required depth at the meter station. When the flow was steady the hook gage was read, the depths were measured, velocity was measured with one meter (requiring 7 to 14 observations of 50 to 100 seconds each), depths were again measured, the hook gage was again read; then the velocity was measured with the other meter, the depths were measured again and the hook gage was read; the gate was then changed, and two more discharge measurements made. In some of the experiments the depth was changed without changing the discharge.

The quantity of water used was so small that the velocity of approach to the weir was small and the head could be read with accuracy. The depths at the meter station were measured with a sharp-edged thin scale, made for the purpose, marked to hundredths of a foot. For depths less than 0.7 foot the depth could be read to 0.002 foot for a depth of 1 foot. The surface was rough, being a succession of small, irregular, dashy, waves.

FLUME, WEIR, METERS, ETC.

The following table gives the results of 36 meter-discharge measurements and the corresponding weir discharges, arranged in the order of depths of water at the meter station for series 1, with smooth sides and bottom. The weir discharge is taken from "The Use of Water in Irrigation," by Elwood Mead and C. T. Johnson, United States Department of Agriculture, Table 1, page 49. This table is computed from the formula $Q=3.3\frac{2}{3}\,Lh^{\frac{3}{2}}$, L being the length of weir crest in feet, h the head in feet, and Q the discharge in cubic feet per second:

Current meter and Cippoletti weir discharge comparisons in 4-foot wooden flume with smooth bottom and sides.

No.	Date.	Weir head.	Depth.	Discharge Q_w	Velocity V_w	Discharge Q_f	Discharge Q_p	$\frac{Q_w-Q_f}{Q_w}$	$\frac{Q_w-Q_p}{Q_w}$	Discharge Q_f	$\frac{Q_w-Q_f}{Q_w}$
	1903.	*Feet.*	*Feet.*	*Cub. ft.*	*Feet.*	*Cub. ft.*	*Cub. ft.*	*Per cent.*	*Per cent.*	*Cub. ft.*	*Per cent.*
1..	Feb. 14	0.270	0.286	0.945	0.822	.....	0.981	.....	− 3.8	.....	.....
2..	Jan. 22	.250	.400	.842	.532	0.862	.....	−2.4	.....	0.852	− 1.2
3..	....do...	.250	.400	.842	.532	.....	.868	.....	− 3.1	.....	.....
4..	Feb. 14	.270	.400	.945	.588	.947	.....	−0.2	.....	.....	.....
5..	....do...	.270	.400	.945	.585	.....	.954	.....	− 1.0	.....	.....
6..	Jan. 22	.324	.460	1.242	.677	1.253	.....	−0.8	.....	1.244	− 0.1
7..	....do...	.324	.460	1.242	.677	.....	1.231	.....	+ 0.9	.....	.....
8..	Jan. 24	.205	.461	.625	.336	.....	.606	.....	+ 3.0	.....	.....
9..	....do...	.205	.462	.625	.336	.635	.....	−1.6	.....	.598	+ 4.3
10..	Jan. 23	.362	.477	1.466	.758	.....	1.473	.....	− 0.5	.....	.....
11..	Jan. 26	.185	.499	.536	.267	.....	.530	.....	+ 1.0	.....	.....
12..	....do...	.185	.499	.536	.267	.536	.....	0.0	.....	.494	+ 7.8
13..	....do...	.187	.502	.545	.270	.....	.541	.....	+ 0.8	.....	.....
14..	....do...	.187	.510	.545	.270	.538	.....	+1.3	.....	.494	+ 9.4
15..	....do...	.187	.516	.545	.263	.566	.....	−3.8	.....	.521	+ 4.4
16..	Jan. 23	.423	.535	1.853	.860	1.884	.....	−1.7	.....	1.862	− 0.5
17..	....do...	.423	.537	1.853	.858	.....	1.883	.....	− 1.6	.....	.....
18..	Jan. 24	.170	.561	.472	.208	.459	.....	+2.8	.....	.384	+18.6
19..	....do...	.170	.561	.472	.208	.....	.406	.....	[b]+13.8	.....	.....
20..	Jan. 26	.238	.581	.782	.334	.782	.....	+0.0	.....	.732	+ 6.4
21..	Feb. 14	.410	.581	1.768	.756	.....	1.798	.....	− 1.7	.....	.....
22..	Jan. 26	.238	.581	.782	.334	.....	.780	.....	+ 0.3	.....	.....
23..	Feb. 14	.410	.582	1.768	.756	1.725	.....	+2.4	.....	.....	.....
24..	Jan. 26	.187	.585	.545	.242	.572	.....	[a]−5.0	.....	.516	+ 5.2
25..	Jan. 23	.499	.603	2.374	.966	.....	2.327	.....	+ 1.9	.....	.....
26..	....do...	.499	.603	2.374	.966	2.448	.....	−3.1	.....	2.424	− 2.1
27..	Jan. 26	.258	.609	.883	.359	.869	.....	+1.6	.....	.827	+ 6.3
28..	Jan. 23	.566	.664	2.868	1.076	2.916	.....	−1.7	.....	2.889	− 0.8
29..	....do...	.566	.665	2.868	1.076	.....	2.857	.....	+ 0.4	.....	.....
30..	....do...	.658	.745	3.595	1.199	.....	3.440	.....	[a]+ 4.3	.....	.....
31..	....do...	.660	.748	3.610	1.198	3.729	.....	−3.3	.....	3.693	− 2.3
32..	Feb. 14	.555	.770	2.785	.891	.....	2.752	.....	+ 1.2	.....	.....
33..	....do...	.557	.770	2.800	.903	2.678	.....	+2.1	.....	.....	.....
34..	Jan. 23	.362	.841	1.466	.758	1.495	.....	−2.0	.....	1.483	− 1.2
35..	Feb. 14	.722	.997	4.131	1.027	4.050	.....	+2.0	.....	.....	.....
36..	....do...	.732	1.010	4.217	1.036	.....	4.212	.....	+ 0.1	.....	.....

[a] Depth varied during measurement. [b] Not velocity enough in some parts of section to start meter.

NOTE.—The Fteley meter discharge is computed from observations 0.2 foot below surface and 0.2 foot above bottom. The Price meter discharge is computed from observations 0.15 foot below surface and 0.15 foot above bottom.

Q_w=weir discharge. Q_f=Fteley meter discharge. Q_p=Price meter discharge. Computed from rating November 2, 1901. V_w=velocity at meter station computed from Q_w.

The Fteley meter discharges given in column 11 are computed from a rating of this meter November 2, 1901. As it had been used very little it was supposed that its rating had not changed much, but the new rating February 4, 1903, at Chevy Chase, Md., shows a change of about 1 per cent in velocity from 0.6 to 1.2 feet, and a larger change for the smaller velocities. Some of this change is due to increased friction of the meter and some to different conditions in the two ratings (see pp. 94, 95).

With three exceptions, viz, Nos. 19, 24, and 30, the meter discharge computed from the new rating table approximates to the weir discharge. The per cent variation of meter from weir discharge is from −3.8 to +2.4 for the Fteley, and from −3.8 to +3.0 for the Price. Of the three exceptions noted, No. 19 is due to the very small velocity in some parts of the water section, the meter wheel not turning part of the time. The other two exceptions are due to an irregular variation in depth at meter station, as ice formed on the baffle (*G*) and obstructed the flow.

The results of the 26 meter-discharge measurements and corresponding weir-discharge measurements for the mixed gravel bed are given in the following table. On January 28, 29, and 30 the meters were held as close to the bed as possible, the center of the small Price being 0.15 and the Fteley 0.20 foot above bottom. On February 10 both meters were held with centers 0.3 foot above bottom. It is seen that the meter discharge is less than the weir discharge. For the small Price at 0.15 foot from bottom the variation is from +2.8 to +19.2 per cent, and at 0.3 foot from bottom the variation is from +1.3 to +7.6 per cent. For the Fteley meter held 0.2 foot above bottom the variation is from +1.6 to +6.1 per cent, and when held at 0.3 foot above bottom the variation is from −0.7 to −2.5 per cent. A wider range of variation of meter from weir discharge is to be expected in a section of varying depth, for there are shallow parts at each end of section in which the velocity can not be accurately measured.

Current-meter and Cippoletti weir-discharge comparisons in 4-foot flume with mixed-gravel bed.

No.	Date.	Weir head.	Depth.	Q_w	V_w	Q_f	Q_p	$\frac{Q_w-Q_f}{Q_w}$	$\frac{Q_w-Q_p}{Q_w}$
	1903.	*Feet.*	*Feet.*	*Cub. feet.*	*Feet.*	*Cub. feet.*	*Cub. feet.*	*Per cent.*	*Per cent.*
1	Jan. 28	0.207	0.381	0.634	0.793		0.606		+ 4.7
2	Jan. 30	.270	.439	.946	.954		.864		+ 8.7
3	Jan. 28	.207	.509	.634	.515	0.624		+1.6	
4	....do...	.207	.515	.634	.511		.518		+18.4
5	Feb. 10	.417	.642	1.813	1.081	1.826		−0.7	
6	....do...	.417	.642	1.813	1.081		1.789		+ 1.3
7	Jan. 30	.411	.647	1.775	1.044		1.620		+ 8.7
8	Jan. 28	.264	.712	.914	.470	.892		+2.4	
9	....do...	.264	.713	.914	.470		.886		+ 2.9
10	Jan. 29	.271	.723	.950	.480	.900		+5.3	
11	....do...	.271	.724	.950	.480		.768		+19.2
12	Feb. 10	.500	.764	2.381	1.111	2.440		−2.5	
13	....do...	.500	.766	2.381	1.111		2.200		+ 7.6
14	Jan. 29	.346	.828	1.370	.571		1.150		+16.1
15	....do...	.346	.830	1.370	.571		1.168		+14.7
16	....do...	.346	.830	1.370	.571	1.286		+6.1	
17	....do...	.458	.840	2.087	.860		1.847		+11.5
18	....do...	.458	.842	2.087	.856	1.985		+4.8	
19	Jan. 30	.547	.844	2.724	1.112		2.542		+ 6.7
20	....do...	.553	.849	2.774	1.082	2.667		+3.8	
21	Feb. 10	.569	.875	2.890	1.124	2.923		−1.2	
22	....do...	.569	.875	2.890	1.124		2.726		+ 5.6
23	Jan. 29	.458	1.008	2.087	.670	1.986		+4.8	
24	....do...	.458	1.010	2.087	.671		1.776		+14.9
25	Feb. 10	.686	1.050	3.826	1.170	3.916		−2.3	
26	....do...	.691	1.050	3.867	1.170		3.728		+ 3.6

Discharge on January 28, 29, and 30, computed from velocity observations, 0.15 foot above bottom and 0.15 foot below surface with Price meter and 0.2 foot above bottom and below the surface with Fteley meter.

Discharge on February 10, computed from observations, 0.3 foot above bottom, and 0.15 foot below surface with Price meter and 0.2 foot below surface with Fteley meter.

It appears that the roughness of the bed has a greater influence on the Price than on the Fteley meter. In the case of the Price meter the variation of meter from weir measurement increases as the velocity decreases.

The following table gives the results of 11 comparisons of meter and weir discharge for a gravel and stony bed. The center of each meter was held 0.30 foot above bottom, except at the sides, where the depth was not sufficient to permit it. The variation for the Price meter is from 0.0 to + 8.4 per cent (with the exception of No. 1), and for the Fteley the variation is from − 5.6 to + 3.9 per cent.

Current-meter and Cippoletti weir discharge comparisons in 4-foot flume with gravel and stone bed.

No.	Date.	Head.	Depth.	Q_w	V_w	Q_f	Q_p	$\frac{Q_w-Q_f}{Q_w}$	$\frac{Q_w-Q_p}{Q_w}$
	1903.	*Feet.*	*Feet.*	*Cub. ft.*	*Feet.*	*Cub. ft.*	*Cub. ft.*	*Per cent.*	*Per cent.*
1	Feb. 12	0.264	0.442	0.914	0.956	.....	0.791	.....	[a] +13.4
2	....do...	.676	.720	3.743	1.906	.....	3.426	.....	+ 8.4
3	....do...	.676	.720	3.743	1.906	3.627	.....	+3.1	.....
4	....do...	.470	.764	2.170	1.018	2.292	.....	−5.6	.....
5	....do...	.470	.764	2.170	1.018	.....	2.171	.....	+ 0.0
6	....do...	.675	.821	3.735	1.590	3.784	.....	−1.3	.....
7	....do...	.675	.827	3.735	1.576	.....	3.457	.....	+ 7.4
8	....do...	.590	.928	3.052	1.096	.....	3.009	.....	+ 1.1
9	....do...	.590	.928	3.052	1.096	2.965	.....	+2.9	.....
10	....do...	.676	1.057	3.743	1.132	3.595	.....	+3.9	.....
11	....do...	.676	1.063	3.743	1.124	.....	3.543	.....	+ 5.3

[a] Depth too small to measure velocity at edges.
Small Price meter discharge, computed from observations, 0.30 foot above bottom and 0.15 foot below surface.
Fteley meter discharge, computed from observations, 0.3 foot above bottom and 0.2 foot below surface.

It appears from these tests that the method in use of measuring the flow of small streams with current meter held close to the bed and to the surface, and using as the mean velocity the half sum of top and bottom velocity, gives results agreeing closely with those given by a weir if the bed is smooth, depth from 0.4 to 1 foot and velocities of 0.4 to 1.5 feet per second. For a gravely bed the meter should be held with its center 0.25 to 0.35 foot above the bed.

STUDIES OF ACCURACY OF MEASUREMENT OF STREAM FLOW.

The object of the following studies was to determine, or at least throw light upon, the accuracy of river-gaging work, and to improve the methods. The Cornell University experiments show the degree of accuracy where conditions are artificial and controllable. But it is impossible to control the flow of rivers for the purpose of measurement, and it is impracticable to construct measuring weirs in order to test the accuracy of current-meter work; so some criterion of accuracy of work must be adopted other than the standard weir. The writer has adopted the two following criteria: (1) He has compared velocity obtained at chosen stations by different methods with that obtained

from vertical velocity curves at the same place. (2) He has compared discharge obtained by the common methods with that obtained from the station rating curve. In other words, the departure or variation of any measured discharge from the mean discharge for that river stage is a measure of its accuracy. It may be asked why not use the vertical-velocity-curve method in all cases. For two reasons: (1) It requires too much time and expense, and (2) the stage of some rivers changes too rapidly to permit its use in all cases. A change of a foot or more per hour is common on some streams. The velocity from one bank of a stream to the other usually varies so much that it is necessary to measure it in many verticals, and if it is measured in more than one or two points in each vertical so much time is consumed that there may be a considerable change in stage and a large error introduced. It is very desirable that the point of mean velocity (distance below water surface) be known, so that the meter can be held there and the velocity in a vertical found in one observation. The vertical velocity curves show the position and amount of the mean velocity, hence their importance. If the shape of these curves were constant and known in advance, the point of mean velocity would be known. The shape varies, however, with roughness of bed, shape and depth of channel, and river stage.

Many of the gaging stations are located at bridges to avoid the expense of a cable equipment. The piers interfere with the natural flow, thus making accurate discharge measurements near them difficult. These studies are expected to throw light on the relative accuracy of discharge measured from a bridge with piers and a station where the flow is unobstructed.

The following streams were selected for these studies: Chittenango Creek, a small stream with unobstructed flow at the carriage bridge in Chittenango Village, N. Y.; Chenango and Susquehanna rivers, streams of moderate size, with flow unobstructed at bridges in Binghamton, N. Y.; Oswego River, of something over moderate size, with unobstructed flow at the cable station near Battle Island, N. Y. Skaneateles Lake outlet, on which is a measuring weir; and the flume of the Truckee River General Electric Company, at Farad, California. These stations are described and illustrated in the discussion of the work done at each.

WORK ON CHITTENANGO CREEK AT CHITTENANGO, N. Y.

This stream rises in the southwestern part of Madison County in the east-central part of New York State, and flows northwest into Oneida Lake. Thence its waters find their way, through Oneida and Oswego rivers, to Lake Ontario. On it are Cazenovia Lake and Erieville reservoir, which regulate its flow to some extent.

The station is at Main Street Bridge, Chittenango, about 18 miles

from the mouth of the creek. The bridge is at a bend in the stream, and at an angle of 60° with the creek. The bed is mixed coarse and fine gravel. The cross section of the stream, on July 15, 1901, is shown in fig. 34. The gage is a board marked to feet and tenths,

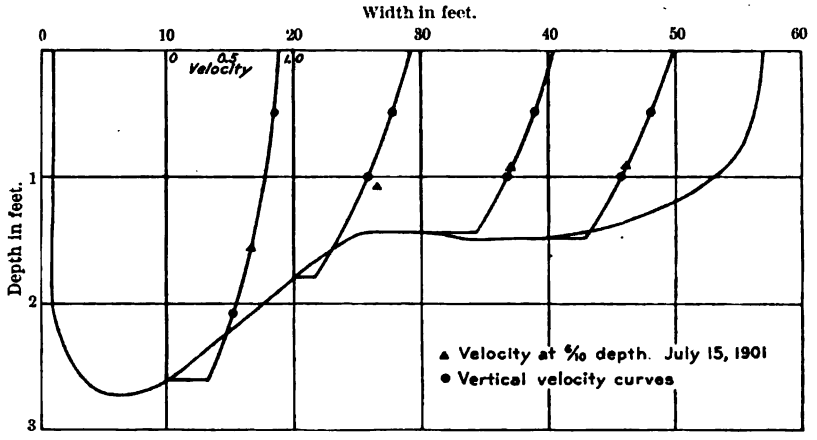

FIG. 34.—Cross section of Chittenango Creek.

fastened vertically to the right abutment on the upstream side of the bridge.

Measurements were made from the upper side of the bridge on July 15, 1901, with small Price meter No. 377, and April 2, 1902, with small Price meter No. 375. Some of the results are given in the following tables. Fig. 34 shows the cross section at the time of the first

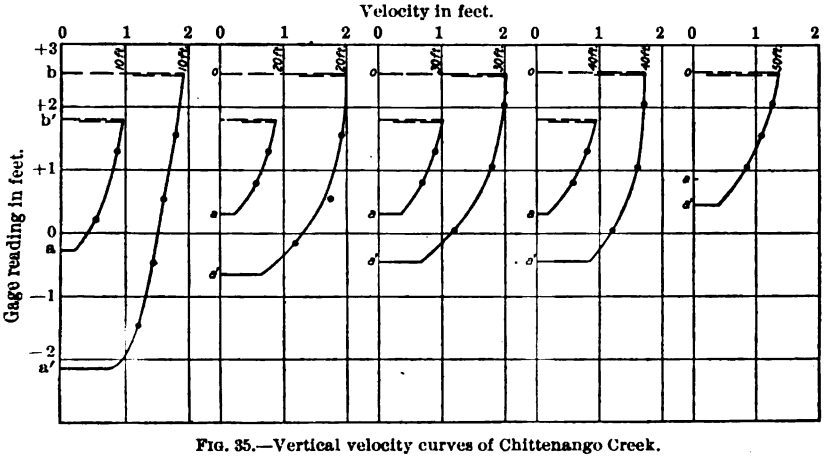

FIG. 35.—Vertical velocity curves of Chittenango Creek.

measurement, the velocity curve at six-tenths depth, and vertical velocity curves at 10, 20, 30, and 40 feet, respectively, from the initial point. Fig. 35 shows the velocity curves obtained in these verticals on July 15, 1901, and April 2, 1902. The two sets of vertical velocity curves are plotted side by side, so as to compare curves at same places

for different stream stages. Between these dates a flood scoured out the bed of the stream at this place, making the increase in depth much greater than that shown by the change of stage. b b′ is the change in stage, a change from a gage reading of 1.80 feet to 2.33 feet. a a′ shows the increase in depth due to scour of bed. The two curves at any point are seen to resemble each other very closely. The mean velocity at six-tenths depth is seen to be somewhat less than that obtained from the vertical velocity curves for the first set of measurements and the reverse is true for the second set of measurements—that is, the thread of mean velocity has lowered somewhat as the depth of water increased. On April 2 the discharge obtained from the vertical velocity curves was 2.9 per cent less than that obtained by the six-tenths depth method. Not enough vertical velocity curves were taken July 15 to enable the discharge to be computed, but from those that were taken, one can see that it would be slightly greater than that by six-tenths depth method. For some intermediate stage between the dates of these measurements the discharges by these two methods will agree.

Results of discharge measurements of Chittenango Creek, Chittenango Village, N. Y.

Date.	Method used.	Gage readings.	Area.	Mean velocity.	Discharge.	Meter and rating.
1901.		*Feet.*	*Square feet.*	*Feet.*	*Cubic feet.*	
July 15	$\frac{6}{10}$ depth	1.80	85	0.63	53	No. 37
1902.						
Apr. 2	do	2.33	151	1.40	2.12	No. 37
Apr. 2	Vertical velocity curves	2.33	151	1.36	2.06	Ice.

Vertical velocity curve data, Chittenango Creek, Chittenango, N. Y.

Date.	10 feet from initial point.				20 feet from initial point.			
	Vel. $\frac{6}{10}$ D.	Vel. v. v. c.	D.	d÷D.	Vel. $\frac{6}{10}$ D.	Vel. v. v. c.	D.	d
1901.								
July 15	0.678	0.695	2.6	0.59	0.667	0.660	1.5	
1902.								
Apr. 2	1.460	1.520	4.7	.55	1.708	1.670	3.2	

Date.	30 feet from initial point.				40 feet from initial point.			
	Vel. $\frac{6}{10}$ D.	Vel. v. v. c.	D.	d÷D.	Vel. $\frac{6}{10}$ D.	Vel. v. v. c.	D.	d
1901.								
July 15	0.678	0.710	1.5	0.54	0.644	0.670	1.5	
1902.								
Apr. 2	1.657	1.750	3.0	.57	1.494	1.470	3.1	

WORK ON SKANEATELES LAKE OUTLET.

Skaneateles Lake is fifth in size of the group of lakes in central New York that drain into the Seneca River and thence into Lake Ontario. The water surface of the lake has an area of 12.75 square

miles, and the drainage area above the foot of the lake is 73 square miles.

In 1893 the city of Syracuse constructed a masonry dam across the outlet at the foot of the lake, converting the lake into a storage reservoir, and since that time has used a considerable part of the discharge from the watershed for a municipal water supply.

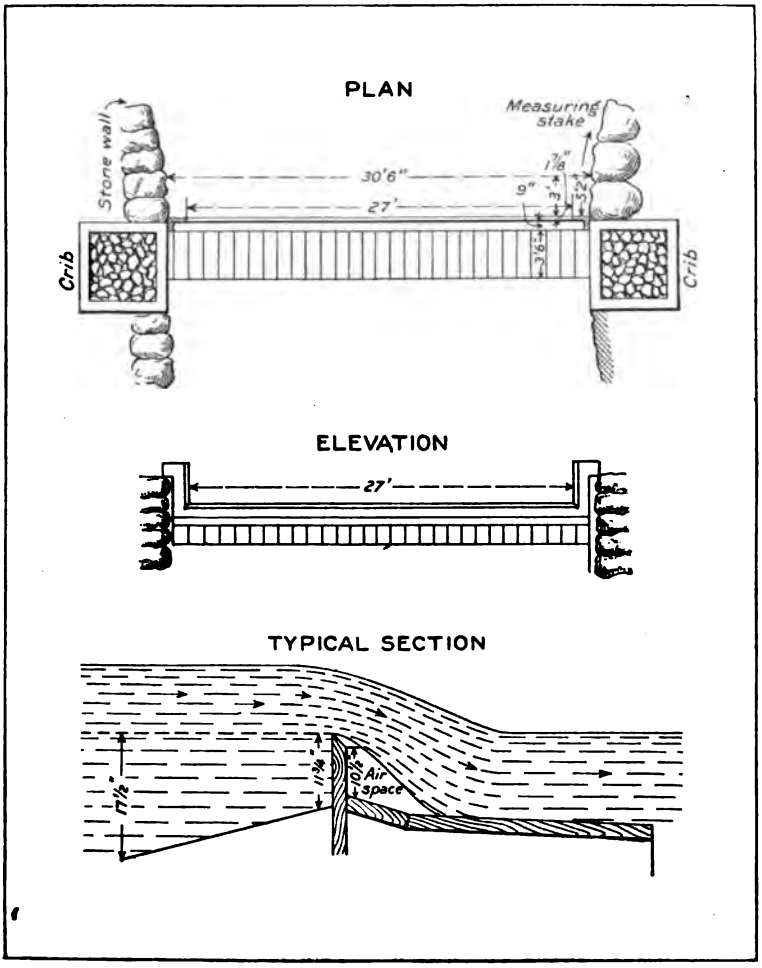

FIG. 36.—Plan of Willowglen weir.

In 1894 what is known as the New Willowglen measuring weir was built on the outlet by the city of Syracuse, about 1.5 miles from the foot of the lake. The plan, elevation, and section of this weir are shown in fig. 36. It has a sharp crest 27 feet in length, with end contractions. The head is measured from a measuring post 5.2 feet upstream from the crest. The channel above the weir is straight and

has an average depth at the weir of 1.5 feet. The velocity of approach varies from 0 to about 2 feet per second.

Francis's formula for flow over sharp-crested weirs, with complete contractions, has been used to compute the flow over this weir. It is not strictly applicable, however, as the depth of water below the crest and the distance from the ends of the weir to sides of channel approach are too small to permit the contraction to be complete. The discharge is probably slightly greater than computed by Francis's formula.

Three measurements of the discharge of this outlet were made from a temporary bridge August 29, 1901, with small Price meter No. 377, about 500 feet above the measuring weir in a straight reach of the stream.

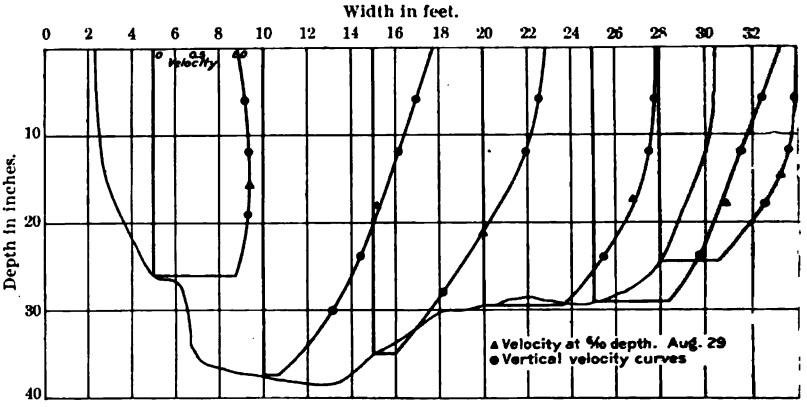

FIG. 37.—Vertical velocity curves, Willowglen, N. Y.

In two of these the mean velocity was obtained from measurements at six-tenths depth; in the third it was found from vertical velocity curves. Fig. 37 shows the cross section of the stream at the place of measurement, also the curve of velocity at six-tenths depth and the vertical velocity curves. The bed is smooth, being clay, sand, and fine gravel. The results are given in the following table:

Results of discharge measurements on Skaneateles Lake outlet, Willowglen, N. Y.

Date.	Method used.	Head on weir.	Section area.	Mean velocity.	Discharge by meter.	Discharge by weir.	$\frac{Q_w - Q_m}{Q_w}$
1901.		*Inches.*	*Sq. ft.*	*Feet.*	*Sec. ft.* Q_m	*Sec. ft.* Q_w	*Per cent.*
Aug. 29	$\frac{6}{10}$ D	$11\frac{3}{4}$	68.9	1.272	87.7	[a]88.86	+1.3
Aug. 29	V. V. curves	$12\frac{1}{16}$	68.9	1.340	92.3	[a]89.37	−3.3
Aug. 29	$\frac{6}{10}$ D	$12\frac{1}{16}$	68.9	1.334	91.9	[a]89.37	−2.9

[a] Computed from table furnished by R. E. Horton.

The discharge by meter calculated from the vertical velocity curves is seen to be 3.2 per cent greater than that given by the weir formula used. This is about what would be expected, as the contraction not being complete on the crest and ends of the weir, the discharge should be larger than with complete contraction.

The effect of the banks in modifying the shape of the vertical velocity curves is shown by the curves at 5 and 28 feet from the initial point. The effect of bed and bank here is to lower the thread of maximum velocity instead of raise it.

WORK ON TRUCKEE RIVER AT FARAD, CAL.

The headworks and flume of the Truckee River General Electric Company are shown on Pl. VI. Water is taken from the Truckee River at Floriston, Cal., and brought to the power house at Farad in a wooden flume about 10 feet wide, 7 feet deep, and 8,500 feet long. Vertical velocity curves were obtained at two places in this flume, one 590 feet from lower end and 150 feet below a curve, the other 220 feet farther downstream, beyond the influence of the curve.

At the sections of measurement, the sides of flume are smooth. There are four cleats on each side covering the four joints, each cleat 4 by ¾ inches. The mean width at the upstream section was 10.09 feet and mean depth 5.98 feet. At the downstream section the mean width was 10.09 feet and the mean depth 5.96 feet.

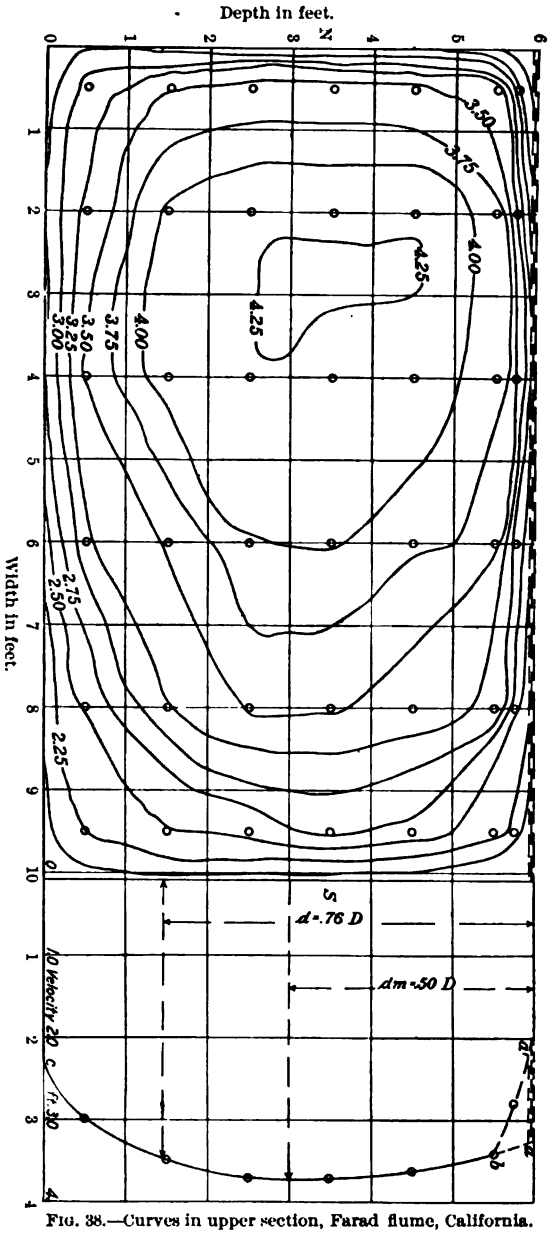

FIG. 38.—Curves in upper section, Farad flume, California.

Velocity measurements were made at six points at each of the six verticals, 0.5, 2.0, 4.0, 6.0, 8.0, and 9.5 feet from north side of flume.

The location of these points, the curves of equal velocity, and the mean vertical velocity curve for upper section are shown in fig. 38. The upper part of the vertical velocity curve, a′ b, is evidently in error.

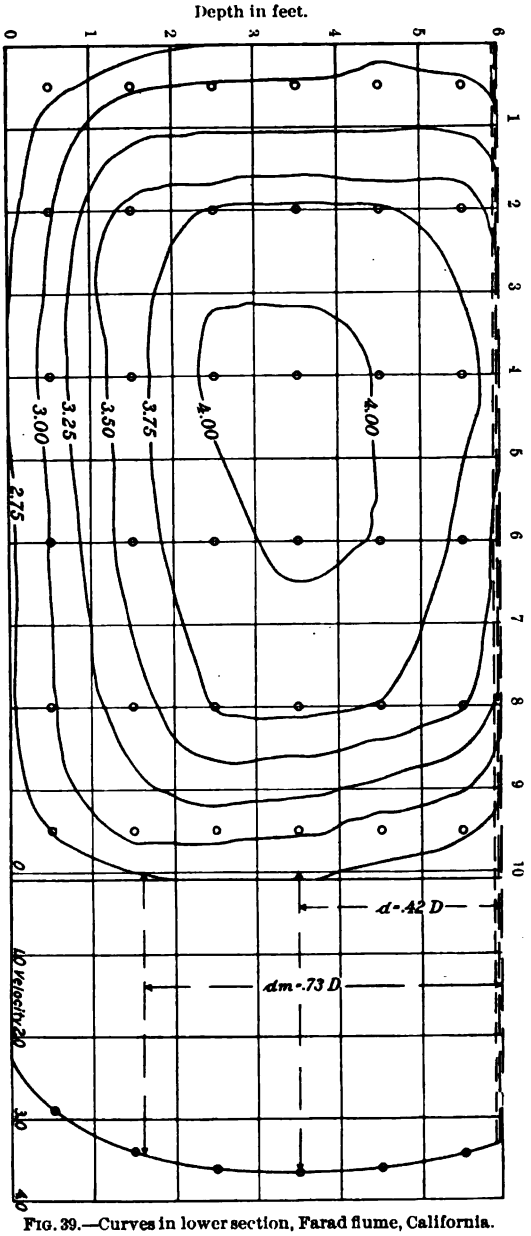

FIG. 39.—Curves in lower section, Farad flume, California.

There was no wind at the time of measurement or any other cause of velocity retardation near the surface. It is due to a peculiarity of this make of meter. For velocities greater than about 2.5 feet per second it indicates too small a velocity when held near the surface. (See p. 90.) The points (centers of the little circles) defining each vertical velocity curve, as well as those on curve a b c, all fall on a well-defined curve. The velocity was very uniform at each point. The thread of mean velocity is at 0.76 of the depth below the surface, and the thread of maximum velocity at 0.50 depth.

Fig. 39 shows the mean vertical velocity curve, and the curves of equal velocity in the lower section. The thread of maximum velocity is near the center of the canal at 0.42 depth, and the thread of mean velocity at 0.73 depth below the surface.

The discharge of the flume at the upper section was 209 second-feet, at the lower 205 second-feet, a difference of about 2 per cent of the latter.

A. TRUCKEE RIVER FLUME.

B. CAR AND CABLE AT OSWEGO RIVER GAGING STATION, BATTLE ISLAND, NEW YORK.

WORK ON CHENANGO RIVER AT BINGHAMTON, N. Y.

This stream rises in Madison County, N. Y., and empties into Susquehanna River at Binghamton, N. Y. The gaging station is at Court Street Bridge, Binghamton. The channel here is straight for

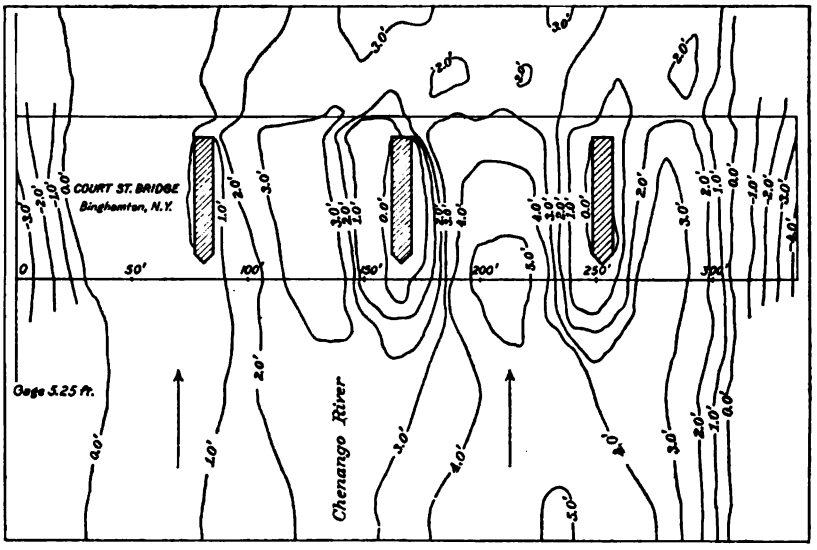

FIG. 40.—Contour of bottom of Chenango River at Court Street Bridge, Binghamton, N. Y.

about 1,000 feet on each side of the station; has a width of about 300 feet at low water and 340 feet at high water; and is broken by three piers. The bed is gravel and cobblestone with large, rough stone

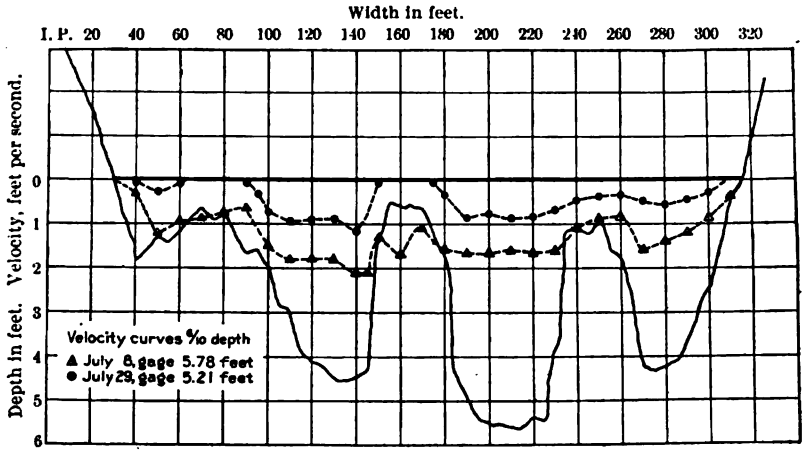

FIG. 41.—Cross section of Chenango River at upper side of Court Street Bridge, Binghamton, N. Y.

around the piers. Fig. 40 shows the contour lines of the stream bed for some distance above and below the bridge, and the location of the piers. It is seen from this figure and fig. 41 that the bed is irregular

as well as rough. This is due to the filling around the piers, and to the riffle a short distance below the piers. The bed is permanent, and there is a good measurable velocity at all river stages. The station is 2,500 feet from the Susquehanna, and is subject to backwater from it at certain stages. Although the channel is broken by three piers, the bridge projects beyond the piers on each side, and the sections of measurement are continuous.

The gage is of the wire-weight type, located on the upper side of the bridge, about half way across it. The initial point is on the west face of the east abutment.

The surface slope of the stream was determined by measuring with a steel tape and 3-pound lead from points of known elevation at the station and on Ferry Street Bridge, 1,500 feet upstream from it.

Velocity measurements were made from the upper and lower sides of the bridge, with a view to comparing discharges and vertical velocity curves on each side. On the lower side, however, the piers produced so much disturbance of the water (eddying) that good results could not be obtained, especially for the higher stages.

The bed being rough, it must be expected that there will be some discrepancies in depth, as shown by the vertical velocity curves. The instrument, when on bottom, may rest on top of a stone, or may slip down between two of them, and thus a movement of a few inches show a difference of 0.2 to 0.4 of a foot in depth. If the meter could be held rigidly with a rod its position would always be accurately known, but when held with a cable, as it necessarily must be from a high bridge, there is always some uncertainty as to its exact location, and there will be discrepancies in different measurements of the depths. The soundings were carefully made twice at each side of the bridge when the river was low. These, and the change in the river stage, were used in computing areas of cross section.

Twenty-two discharge measurements have been made at this station. From July 2, 1901, to August 15, 1902, nineteen of these were made by the six-tenths-depth method, and three by Harlacher's method. For obtaining discharge, the velocity was measured in verticals 10 feet apart, except near the center of each bridge span, where there was little change in velocity, and where they are 15 to 20 feet apart. Each observation was usually for two periods of fifty seconds each. Fifteen sets of vertical velocity curves were obtained, each set containing usually 8 curves, at distances 100, 120, 140, 190, 210, 230, 275, and 290 feet from the initial point. These curves are not used in computing discharge, as they were not taken sufficiently close together near the piers to enable mean velocity in section to be computed, but they are used to show amount and position of mean velocity in a vertical with which to compare velocity at six-tenths depth.

The results of the discharge measurements are given in the following table:

Discharge measurements, Chenango River, Binghamton, N. Y.

No.	Date.	Method.	Gage reading.	Area section.	Discharge.	Mean velocity.	Place.
	1901.		*Feet.*	*Sq. ft.*	*Cub. ft.*	*Feet.*	
1	July 2	0.6 depth	5.64	689	848	1.23	(a)
2	July 8	.....do	b 5.78	764	1,119	1.46	(c)
3	July 11	.....do	.....	1,666	768	0.46	(d)
4	July 9	.....do	5.71	617	942	1.53	(a)
5	July 29	.....do	5.21	602	405	0.61	(d)
6	....do	.....do	5.21	469	425	0.90	(a)
7	Aug. 19	.....do	5.49	681	577	0.85	(d)
8	....do	.....do	5.48	547	566	1.04	(a)
9	Oct. 19	.....do	e 5.82	775	927	1.20	(d)
10	....do	.....do	e 5.81	646	987	1.53	(d)
	1902.						
11	Mar. 27	.....do	8.15	1,384	4,201	3.04	(d)
12	Mar. 28	.....do	8.21	1,489	4,377	2.94	(a)
13	Mar. 29	.....do	8.75	1,590	5,205	3.27	(d)
14	July 1	Harlacher's	8.44	1,564	4,439	2.83	(a)
15	....do	0.6 depth	8.41	1,534	4,815	3.14	(a)
16	July 3	Harlacher's	7.16	1,155	2,611	2.26	(a)
17	....do	0.6 depth	7.16	1,155	2,688	2.33	(a)
18	July 15	.....do	f 6.56	995	2,093	2.13	(a)
19	Aug. 2	Harlacher's	f 9.10	1,795	5,426	3.02	(a)
20	Aug. 3	0.6 depth	f 9.04	1,775	5,543	3.12	(a)
21	Aug. 14	.....do	f 6.20	877	1,608	1.83	(a)
22	Aug. 15	.....do	f 6.08	841	1,341	1.48	(a)

a Upper side Court Street Bridge.
b This reading is probably wrong: it should be 5.88.
c Cable above bridge.
d Lower side Court Street Bridges.
e Reading too large on account of backwater.
f Corrected by —0.09 foot for stretch of wire gage.

The gage reading of measurement No. 2 is evidently in error. The velocities are all larger than the corresponding ones of No. 10. The correct reading is probably 5.88; that is, there is a mistake of 0.1 foot.

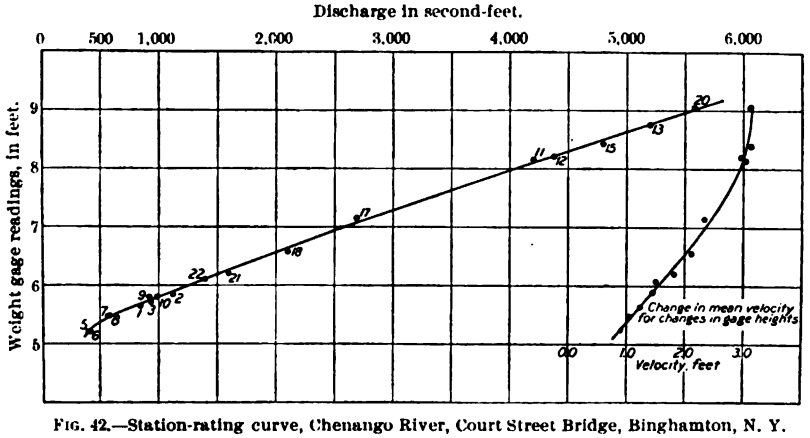

FIG. 42.—Station-rating curve, Chenango River, Court Street Bridge, Binghamton, N. Y.

The quantities in columns 4 and 6 are plotted in fig. 42, which shows the station-rating curve. From this curve the following station-rating table has been prepared:

Station-rating table, Chenango River, Court Street Bridge, Binghamton, N. Y.

Gage reading.	Discharge.	Gage reading.	Discharge.	Gage reading.	Discharge.
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
5.2	415	6.9	2,490	8.6	4,[illegible]
5.3	450	7.0	2,625	8.7	5,1[illegible]
5.4	510	7.1	2,765	8.8	5,[illegible]
5.5	625	7.2	2,905	8.9	5,[illegible]
5.6	755	7.3	3,045	9.0	5,[illegible]
5.7	885	7.4	3,185	9.1	5,[illegible]
5.8	1,015	7.5	3,325	9.2	5,[illegible]
5.9	1,145	7.6	3,470	9.3	6,[illegible]
6.0	1,275	7.7	3,615	9.4	6,[illegible]
6.1	1,410	7.8	3,760	9.5	6,[illegible]
6.2	1,545	7.9	3,905	9.6	6,[illegible]
6.3	1,680	8.0	4,050	9.7	6,[illegible]
6.4	1,815	8.1	4,200	9.8	6,[illegible]
6.5	1,950	8.2	4,350	9.9	6,[illegible]
6.6	2,085	8.3	4,500	10.0	[illegible]
6.7	2,220	8.4	4,650		
6.8	2,355	8.5	4,800		

From the two preceding tables, the following, showing the difference between the measured discharge and the discharge given by the station-rating table, has been prepared:

Comparison of measured and mean discharges, Chenango River, Binghamton, N. Y.

Date.	No.	Gage reading.	Discharge.		Discharge difference.	
			By measure.	By rating curve.		
1901.		*Feet.*	*Second-feet.*	*Second-feet.*	*Cubic feet.*	*Per cent.*
July 2	1	5.64	848	807	+ 41	+ 5.1
July 8	2	[a]5.78	1,119	989	+130	+14.2
July 9	3	5.71	942	1,028	− 86	− 8.3
July 29	5	5.21	405	420	− 15	− 3.5
Do	6	5.21	425	420	+ 5	+ 1.2
August 19	7	5.49	577	613	− 36	− 5.9
Do	8	5.48	566	602	− 36	− 6.0
October 19	9	[b]5.82	927	1,041	−114	−10.9
Do	10	[b]5.81	987	1,028	− 41	− 4.0
1902.						
March 27	11	8.15	4,201	4,275	− 74	− 1.7
March 28	12	8.21	4,377	4,365	+ 12	+ 0.3
March 29	13	8.75	5,205	5,175	+ 30	+ 0.6
July 1	15	8.41	4,815	4,665	+150	+ 3.2
July 3	17	7.16	2,688	2,849	−161	− 5.7
July 15	18	6.56	2,098	2,031	+ 67	+ 3.3
August 3	20	9.04	5,543	5,612	− 69	− 1.2
August 14	21	6.20	1,608	1,545	+ 53	+ 3.4
August 15	22	6.08	1,341	1,383	− 42	− 3.0
1903.						
July 1	14	8.44	[c]4,439	4,710	−271	− 5.7
July 3	16	7.16	[c]2,611	2,849	−238	− 8.3
August 2	19	9.10	[c]5,426	5,705	−279	− 4.9

[a] Should probably be 5.88.
[b] Reading too large on account of backwater.
[c] By Harlacher's method.

Omitting No. 2 on account of a probable error in the gage reading, and No. 9 on account of the influence of back water, we see that the variation of the discharges from the mean discharge is from −8.3 to +5.1 per cent.

The 15 vertical velocity curves for each of the 8 verticals are made up from velocity observations at from 3 to 5 points in each vertical.

the extreme ones being about 0.5 foot above bottom and 1 foot below the surface. Each observation covered 4 periods of 25 seconds each. The nearest whole number of revolutions in each period was recorded except the last one, in which the nearest tenth was also recorded. The velocities computed from these observations were plotted on section paper, and a smooth curve drawn to represent the mean vertical velocity curve. These points gave, as a rule, a well-defined curve except near the bottom, where the bed was rough. From these curves the velocity at each two-tenths of the depth and at the surface was taken and tabulated. The data for each vertical was grouped, and

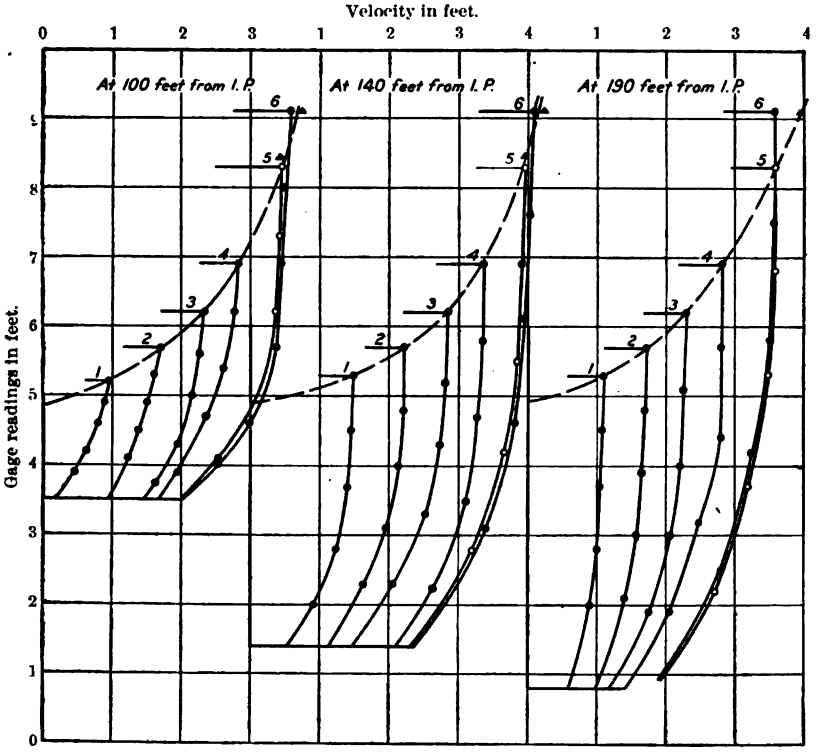

FIG. 43.—Groups of vertical velocity curves, Chenango River, Binghamton, N. Y.

those for each two-tenths of depth and surface were combined for the curves in which the gage heights did not differ by more than 0.5 foot. The resulting six sets of vertical velocity curves on the upper side, and three sets on the lower side of the bridge are shown on figs. 43 and 44. These sets of curves should now be studied in connection with figs. 40 and 41, which show position of piers and irregularity of bed. Perhaps the first thing that is noticed is the similarity—with one exception—of the curves in any group. Curve No. 6 in each group is different from the others, and indicates a disturbance of the ordinary conditions. The reduced surface velocity indicates back-

water, but as no slope measurements were taken that afternoon it can not be definitely stated that this is the cause. Another noticeable feature is the falling of all the surface velocities except No. 6 on a well-defined curve which resembles a parabola. Another is that the maximum velocity is at or near the surface. Another, that as the depth decreases and as the velocity decreases, the vertical velocity curve approaches a straight line. For shallow depths this line is more inclined than for greater depths.

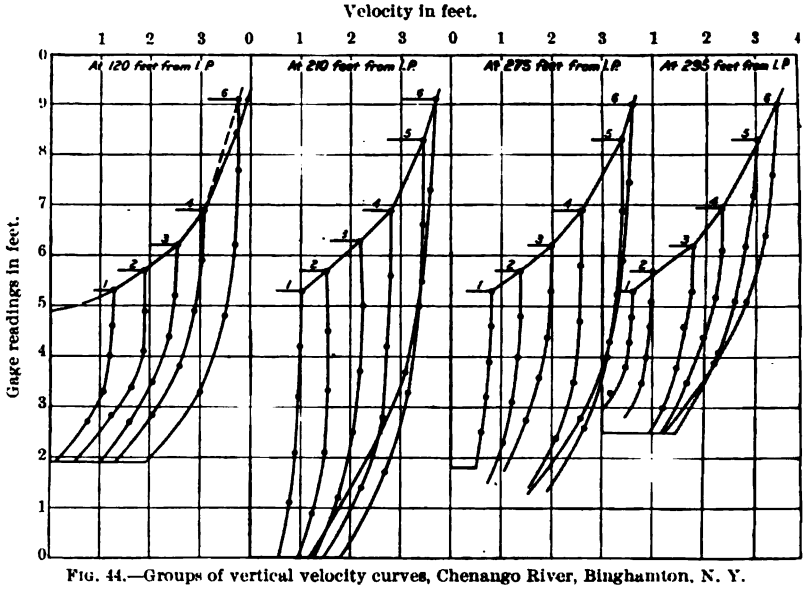

FIG. 44.—Groups of vertical velocity curves, Chenango River, Binghamton, N. Y.

The lower parts of these curves are not well defined on account of roughness of bed; the depths varied and the velocity was irregular and local. The following table shows the change in mean velocity in verticals for changes in river stage:

Table showing change in mean velocity in verticals with changes in river stage, Chenango River, Court Street Bridge, Binghamton, N. Y.

Date.	Gage reading.	Distance of vertical from initial point.							
		100 feet.	120 feet.	140 feet.	190 feet.	210 feet.	230 feet.	275 feet.	290 feet.
1901.	*Feet.*								
July 30	5.21	0.68	0.91	1.11	0.90	0.83	0.75	0.62	0.42
Aug. 20	5.43	.97	1.19	1.31	1.09	.95	.74	.77	.53
July 9	5.64	1.49	1.66	2 09	1.68	1.52	1.41	1.29	.94
Oct. 19[a]	5.80	1.36	1.53	1.78	1.42	1.31	1.15	1.12	.84
1902.									
Aug. 15	6.15	1.72	1.97	2.30	1.90	1.90	.....	1.71	1.40
Aug. 14	6.27	2.17	2.18	2.61	1.98	2.09	1.90	1.77	1.51
July 3	7.18	2.49	2.65	3.10	2.57	2.70	2.30	2.48	2.11
Mar. 28[b]	8.02	3.38	3.74	3.91	3.92	4.05	3.84	.....	.....
Mar. 27[b]	8.14	.....	.....	3.90	4.11	4.12	.....	.....	.....
Mar. 28	8.17	2.93	3.23	3.51	3.19	3.05	2.58	3.06	.....
July 1	8.40	3.04	.....	3.59	3.11	3.08	2.61	3.00	2.42
Mar. 29[b]	8.85	3.60	3.83	4.10	4.19	4.35	.....	.....	.....
Aug. 2[a]	9.07	3.22	3.33	3.65	3.17	3.01	3.15	3.08	2.79

[a] Affected by back water. [b] From lower side of bridge.

These mean velocities and corresponding river stage has been plotted for each vertical, giving curves like that for 140 feet from initial point, as shown in Fig. 45. The velocity does not increase as fast as the river stage. The following table shows the changes in the ratio $\frac{d}{D}$ for river stages:

Table showing change in ratio d÷D in verticals with change in river stage, Chenango River, Court Street Bridge, Binghamton, N. Y.

Gage reading.	Distance of vertical from initial point.							
	100 feet.	120 feet.	140 feet.	190 feet.	210 feet.	230 feet.	275 feet.	290 feet.
Feet.								
5.21	0.54	0.64	0.60	0.57	0.68	0.58	0.56	0.55
5.43	.53	.59	.62	.60	.63	.58	.57	.56
5.64	.56	.65	.61	.61	.63	.60	.58	.53
6.15	.56	.58	.61	.64	.66	.....	.58	.60
6.27	.64	.61	.64	.67	.67	.67	.67	.63
6.64	.57	.59	.67	.62	.65	.66	.64	.57
7.18	.61	.64	.63	.59	.70	.67	.65	.57
8.17	.59	.63	.64	.62	.62	.61	.61	.63
8.40	.67	.....	.70	.63	.67	.63	.67	.55
a 9.07	a .70	a .67	a .69	a .64	a .63	a .67	a .65	a .61
	.586	.616	.636	.616	.657	.625	.614	.577
b 8.02	b .57	b .60	b .63	b .62	b .63	b .53	.....	.....
8.14	.....	.....	b .69	b .59	b .63	.....	.....	.....
8.85	b .57	b .66	b .64	b .66	b .67	.....	.....	.....
.....	.57	.63	.653	.623	.643	.....	.....	.....

a Results affected by backwater and not used in computing means.
b Results from lower side of bridge.

It is seen that this ratio increases somewhat as the depth increases for the shallower depths when the stage is constant, and also increases slightly as the river stage increases. Fig. 45 shows results for the vertical 140 feet from initial point.

The following table shows changes in the ratio $\frac{V_m}{V_s}$ in verticals for changes in river stage, V_s being the velocity 0.5 foot below the surface:

Table showing change in ratio $V_m \div V_s$ in verticals for changes in river stage, Court Street Bridge, Chenango River, Binghamton, N. Y.

Gage reading.	Distance of vertical from initial point.							
	100 feet.	120 feet.	140 feet.	190 feet.	210 feet.	230 feet.	275 feet.	290 feet.
Feet.								
5.21	0.80	0.77	0.83	0.90	0.86	0.80	0.84	0.83
5.43	.90	.80	.83	.94	.85	.78	.86	.81
5.64	.90	.83	.89	.92	.93	.89	.89	.89
a 5.80	a .91	a .85	a .86	a .91	a .90	a .76	a .86	a .91
6.15	.82	.81	.85	.83	.86	.....	.88	.82
6.27	.90	.84	.89	.84	.88	.90	.87	.88
6.64	.89	.81	.90	.89	.88	.82	.90	.88
7.18	.89	.84	.88	.86	.92	.92	.91	.86
8.17	.85	.87	.89	.87	.88	.81	.87	.90
8.40	.89	.....	.92	.89	.91	.84	.90	.82
a 9.07	a .93	a .88	a .89	a .89	a .83	a .88	a .86	a .81
	.871	.821	.876	.882	.897	.833	.880	.855
b 8.02	b .85	b .88	b .90	b .93	b .94	b .91	.....	.....
8.14	.....	.....	b .92	b .93	b .94	.....	.....	.....
8.85	b .86	b .89	b .93	b .94	b .98	.....	.....	.....
	.855	.885	.917	.933	.953	.....	.....	.....

a Results affected by backwater and not used in computing means.
b Results from lower side of bridge.

The values of this ratio and the corresponding gage heights have been platted for each vertical. They all show this ratio to be nearly constant. There is a slight increase of the ratio for increasing river stage. Fig. 45 shows results for vertical 140 feet from initial point.

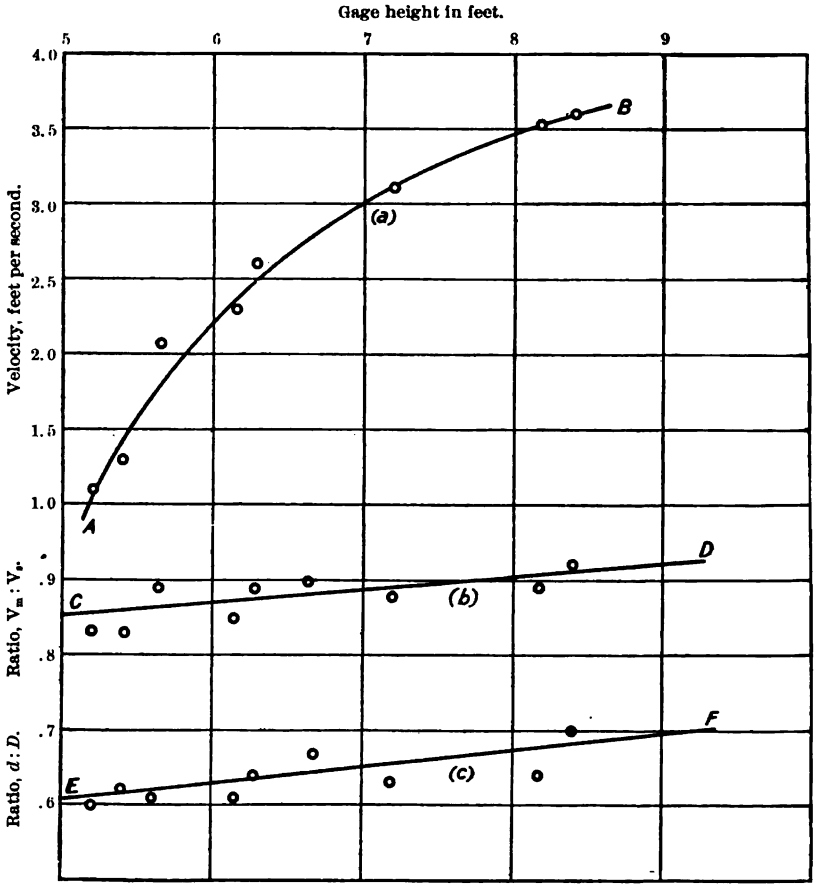

FIG. 45.—Diagram showing results in vertical 140 feet from initial point, Court Street Bridge, Binghamton, N. Y.

The slope of the surface in the vicinity of the gaging station was obtained by measurements with a steel tape from points of known elevation at the gage and at Ferry Street Bridge. The data obtained are given in the following table:

Slope of Chenango River, Binghamton, N. Y., from Court street to Ferry Street Bridge, 1,500 feet.

Date.	Hour.	Gage reading.	Elevations.[a] Court street. C.	Ferry street. F.	Elevation difference. F-C.
1902.		Feet.	Feet.	Feet.	Feet.
March 28	7.30 a. m	8.02	−5.09	−4.72	0.37
Do	10.45 a. m	8.05	−5.06	−4.68	.38
Do	5 p. m	8.32	−4.79	−4.40	.39
March 29	7.30 a. m	8.70	−4.40	−4.01	.39
Do	11.45 a. m	8.91	−4.20	−3.80	.40
July 1	12.14 p. m	8.46	−4.70	−4.31	.39
Do	3.45 p. m	8.40	−4.75	−4.37	.38
July 2	6.55 a. m	8.07	−5.06	−4.68	.38
July 3	1.30 p. m	7.30	−5.82	−5.52	.30
July 15	5.10 a. m	6.68	−6.38	−6.15	.23
Do	1 p. m	6.59	−6.46	−6.22	.24
August 3	6 a. m	8.62	−4.40	−4.09	[b].31
August 4	5.40 p. m	7.88	−5.16	−4.89	[b].27
August 14	2.40 p. m	6.28	−6.74	−6.55	.19
Do	6.35 p. m	6.26	−6.77	−6.57	.20
August 15	5.30 a. m	6.19	−6.83	−6.64	.19
August 16	12.12 p. m	6.09	−6.97	−6.81	.16

[a] Elevations referred to Binghamton City datum. Elevations of B. M. furnished by E. F. Weeks, Assistant City Engineer.
[b] Measurements made during backwater.

NOTE.—From July 3 to July 15, 1902, the gage wire appears to have shortened about 0.09 foot.

These data are plotted in fig. 46, using surface elevations at Court Street Bridge as ordinates, and fall of the surface in 1,500 feet as

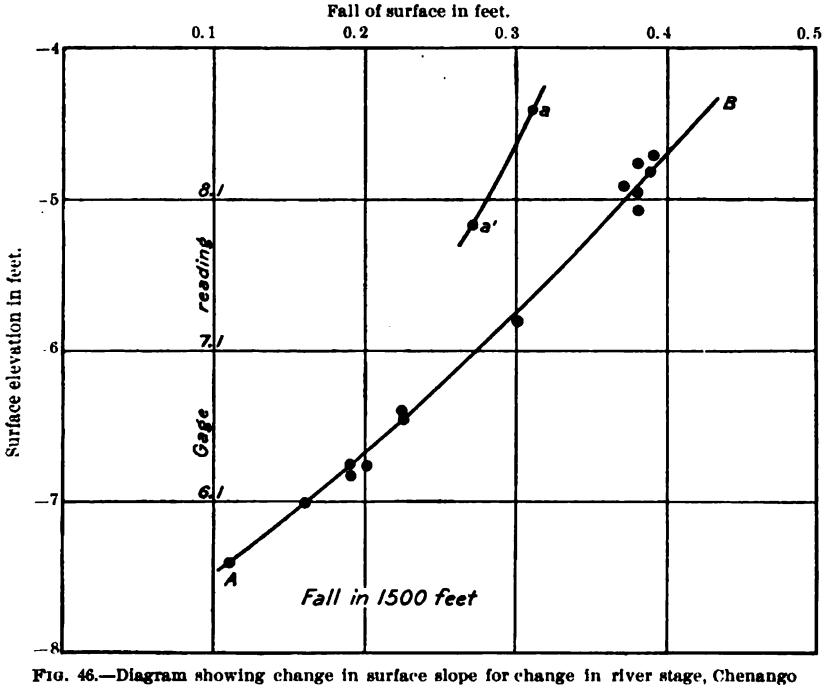

FIG. 46.—Diagram showing change in surface slope for change in river stage, Chenango River, Court Street Bridge, Binghamton, N. Y.

abscissas, giving the curve *A B*. The slope increases with the depth, but the rate of increase is not as great as that of the depth. The points

a and *a'* obtained during the effect of backwater, show that there is less slope and smaller change in slope when there is backwater at a station.

WORK ON SUSQUEHANNA RIVER AT BINGHAMTON, N. Y.

Susquehanna River rises in Otsego County, in the east-central part of New York, and flows southwesterly and southerly through New York, Pennsylvania, and Maryland, into Chesapeake Bay.

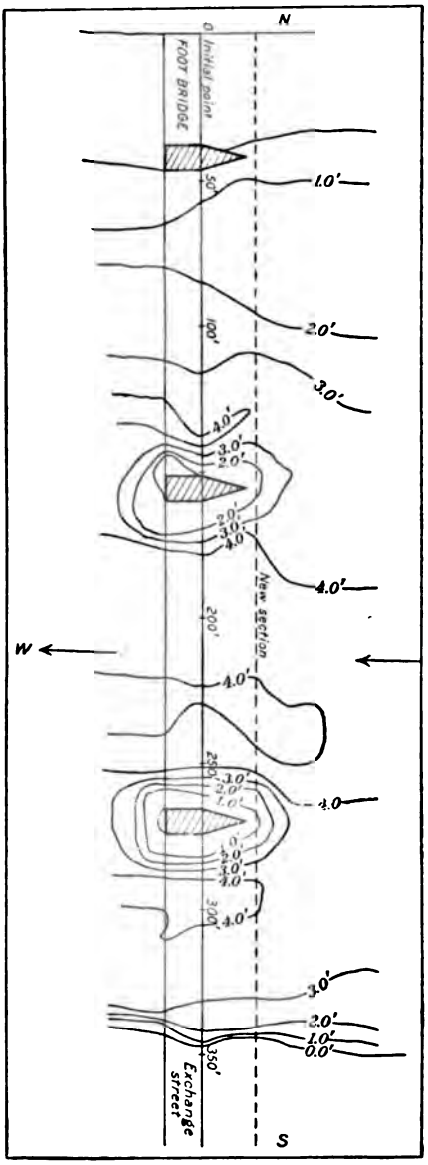

FIG. 47.—Contours of Susquehanna River bed at Exchange Street Bridge, Binghamton, N. Y.

The gaging station, where the work to be described was done, is at Exchange Street Bridge, Binghamton, N. Y. The channel is straight for about 500 feet above and below the station, has a width of about 300 feet at low water and about 450 feet at high water, and is broken by three piers. The bed is gravel and small stone, with large stones filled around the piers. Fig. 47 shows the contour lines of the bottom, and the location of the piers. Fig. 48 shows the cross section on the upper side of the bridge. The shape of the section is good except for the fill around the piers; the bed is permanent, and the velocity large, especially for the higher stages. About 900 feet above the station is the dam of the Binghamton Water Power Company. Its effective head is 6 feet. The thread of the stream strikes the piers somewhat obliquely, producing eddies.

The old wooden bridge at the station was condemned shortly before this station was established, and it was decided to place the gage, which is of the wire-weight form, on Washington Street Bridge, about 1,900 feet below the station. A temporary gage was, however, established at the station, and the water surface referred to it for convenience. The new bridge was erected in the spring of 1902. It has only one

ier, which is about 200 feet from the east abutment. Around it are wo rows of short piles and a quantity of large stone. The upper arts of the old piers have been removed, but the stone filling around hem remains, leaving the river bed irregular and rough.

The method of measuring discharge and of obtaining vertical velocty curves was the same as at the Chenango River station, and need not

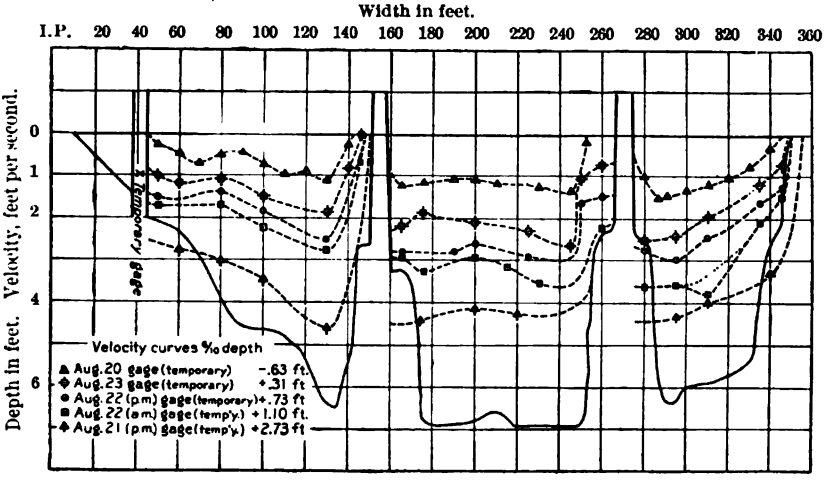
Width in feet.
I.P. 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360
Depth in feet. Velocity, feet per second.
0
1
2
4
6
Temporary gage
Velocity curves ⅒ depth
▲ Aug. 20 gage (temporary) −.63 ft.
◆ Aug. 23 gage (temporary) +.31 ft
● Aug. 22 (p.m.) gage (temporary) +.73 ft
■ Aug. 22 (a.m.) gage (temp'y.) +1.10 ft.
◆ Aug. 21 (p.m.) gage (temp'y.) +2.73 ft

FIG. 48.—Cross section of Susquehanna River at lower side of Exchange Street Bridge, Binghamton, N. Y.

again be described. The current was so swift here that it was difficult to keep the meter in place, and the discharge is computed in some cases from velocity measurements 1 foot below surface, and the ratio of mean to surface velocity found from the vertical velocity curves.

The following table gives the results of the discharge measurements. Those made in 1902 were from the upper side of the new bridge:

Discharge measurements, Susquehanna River, Binghamton, N. Y., Exchange Street Bridge.

No.	Date.	Method used.	Gage reading.	Area.	Discharge.	Mean velocity.	Gage, Wash. st.	Place.
	1901.			*Square feet.*	*Second-feet.*			
1	July 3	0.6 depth	−0.60	891	947	1.06	2.13	(a)
2	July 10	do	− .27	1,020	1,425	1.40	2.28	(a)
3	July 30	do	− .83	847	b608	.72	1.99	(a)
4	Aug. 20	do	− .65	909	942	1.04	2.09	(a)
5	Aug. 20	do	− .63	923	952	1.03	2.10	(c)
6	Aug. 21	do	+2.73	1,989	7,244	3.65	4.63	(a)
7	Aug. 21	Harlacher's	+3.00	2,066	7,803	3.77	4.87	(c)
8	Aug. 22	0.6 depth	+1.10	1,439	3,752	2.61	3.23	(a)
9	Aug. 22	do	+ .73	1,324	2,983	2.25	2.94	(c)
10	Aug. 23	do	+ .31	1,189	2,176	1.83	2.64	(c)
	1902							
11	July 2	Harlacher's	+2.16	1,813	5,386	2.97	4.09	(d)
12	July 2	0.6 depth	+2.09	1,790	5,839	3.26	4.04	(d)
12_2	July 2	Surface ratios	+2.06	1,794	5,680	3.17	4.10	(d)
12_1	July 2	V. V. curves	+2.09	1,790	5,420	3.03	4.04	(d)
13	July 3	Harlacher's	+2.46	1,916	6,559	3.42	4.39	(d)
14	July 4	V. V. curves	+1.87	1,717	5,230	3.28	2.96	(d)
14	July 4	0.6 depth	+1.87	1,717	5,626	3.28	3.86	(d)
15	July 14	do	+ .77	1,320	3,064	2.32	2.96	(d)
16	Aug. 3	Harlacher's	+3.31	2,206	8,156	3.70	5.15	(d)
17	Aug. 3	surface ratios	+3.31	2,187	8,633	3.95	5.15	(d)
18	Aug. 4	Harlacher's	+2.65	1,971	6,599	3.34	4.60	(d)
19	Aug. 4	Surface ratios	+2.65	1,952	6,902	3.53	4.60	(d)
20	Aug. 15	0.6 depth	+ .24	1,140	2,105	1.85	2.60	(d)
21	July 16	do	+ .08	1,103	1,920	1.74	2.50	(d)

a Upper side of old bridge.
b Two small gates in mill closed during measurement.
c Lower side of old bridge.
d Upper side of new bridge.

These discharges and the corresponding temporary gage readings are plotted in fig. 49, which gives the station-rating curve for this

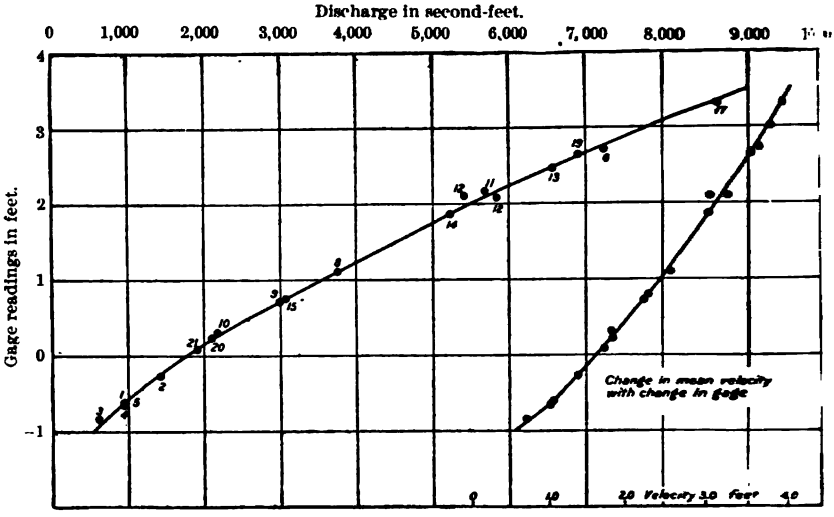

Fig. 49.—Station-rating curve of Susquehanna River at Exchange Street Bridge, Binghamton, N. Y.

station. From this curve the following station-rating table has been prepared:

Station-rating table, Susquehanna River, Exchange Street Bridge, Binghamton, N. Y.

Gage reading.	Discharge.	Gage reading.	Discharge.	Gage reading.	Discharge.
Feet.	*Second-feet.*	*Feet.*	*Second-feet.*	*Feet.*	*Second-feet.*
−1.0	525	0.7	2,955	0.4	[illegible]
− .9	642	.8	3,140	.5	[illegible]
− .8	760	.9	3,335	.6	[illegible]
− .7	879	1.0	3,530	.7	[illegible]
− .6	1,000	.1	3,730	.8	[illegible]
− .5	1,124	.2	3,930	.9	[illegible]
− .4	1,250	.3	4,130	3.0	[illegible]
− .3	1,378	.4	4,330	.1	[illegible]
− .2	1,510	.5	4,530	.2	[illegible]
− .1	1,645	.6	4,730	.3	[illegible]
.0	1,790	.7	4,930	.4	[illegible]
+ .1	1,935	.8	5,130	.5	[illegible]
.2	2,090	.9	5,330	.6	[illegible]
.3	2,252	2.0	5,530	.7	[illegible]
.4	2,415	.1	5,740	.8	[illegible]
.5	2,592	.2	5,950	.9	[illegible]
.6	2,770	.3	6,170	4.0	[illegible]

From this table and the one giving results of discharge measurements the following table, giving the percentage difference between the measured discharge and that shown by the discharge table, has been prepared. With three exceptions this variation is less than 5 per cent. The largest variation is for the smallest river stage, and is due to the closing of the gates of the mill at noon while the measurement was under way. No. 14 is too large, due to the meter being carried downstream, and not being at 0.6 depth. The results by Harlacher's method, No. 7, 11, 13, 16, and 18, indicate, as in the case of the Chenango River station, that the coefficient 0.85 in the formula $Q=0.85\ \Sigma\ \sqrt{s}\ F$ is too small, and should be 0.88.

Comparison of measured and mean discharge, Susquehanna River, Exchange Street Bridge, Binghamton, N. Y.

Date.	No.	Gage.	Discharge.		Discharge difference.	
			By measure.	By curve.	Second-feet.	Per cent.
1901.		*Feet.*	*Second-feet.*	*Second-feet.*		
uly 3	1	−0.60	947	1,000	− 53	− 5.3
uly 10	2	− .27	1,425	1,416	+ 9	+ .7
uly 30	3	− .83	608	725	−117	[a] −16.1
ugust 20	4	− .65	942	940	+ 2	+ .2
Do	5	− .63	952	965	− 13	− 1.3
ugust 21	6	+2.73	7,244	7,155	+ 89	+ 1.2
ugust 22	8	+1.10	3,752	3,730	+ 22	+ .6
Do	9	+ .73	2,983	3,011	− 28	− .9
ugust 23	10	+ .31	2,176	2,268	− 92	− 4.0
1902.						
uly 2	12	+2.09	5,839	5,720	+119	+ 2.1
Do	12_1	+2.09	5,420	5,720	−300	− 5.2
Do	12_2	+2.16	5,680	5,866	−186	− 3.1
uly 4	14	+1.87	5,626	5,270	+356	[b] + 6.6
Do	14_1	+1.87	5,230	5,270	− 40	− .8
uly 14	15	+ .77	3,064	3,084	− 20	− .7
ugust 3	17	+3.31	8,633	8,565	+ 68	+ .8
ugust 4	19	+2.65	6,902	6,965	− 63	− .9
ugust 15	20	+ .24	2,105	2,152	− 47	− 2.2
ugust 16	21	+ .08	1,920	1,906	+ 14	+ .7
ugust 21	7	+3.00	7,803	7,805	− 2	.0
uly 2	11	2.16	5,386	5,866	−480	− 8.2
uly 3	13	2.46	5,660	6,488	+ 71	+ 1.1
ugust 3	16	3.31	8,156	8,565	−409	− 4.8
ugust 4	18	2.65	6,599	6,965	−366	− 5.2

[a] Due to closing the gates in bill at noon.
[b] Due to meter being carried downstream and not at 0.6 depth.

Vertical-velocity-curve data were obtained in 12 verticals as described on p. 117. These were plotted, vertical velocity curves drawn, and the velocities at each one-fifth of the depth and at the surface tabulated. The curves in each vertical in which the change in river stage did not

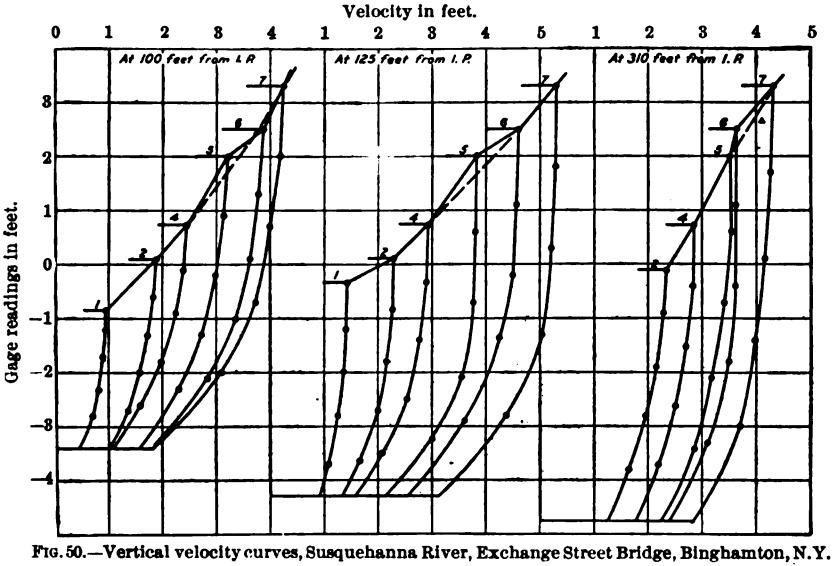

FIG. 50.—Vertical velocity curves, Susquehanna River, Exchange Street Bridge, Binghamton, N.Y.

differ by more than 0.5 foot were then united by combining the velocities at each fifth of the depth. Some of the resulting groups of curves are plotted in figs. 50 and 51. Nos. 4, 5, 6, and 7 were taken from the upper side of the new bridge, which is 18 feet upstream from the upstream face of the old bridge.

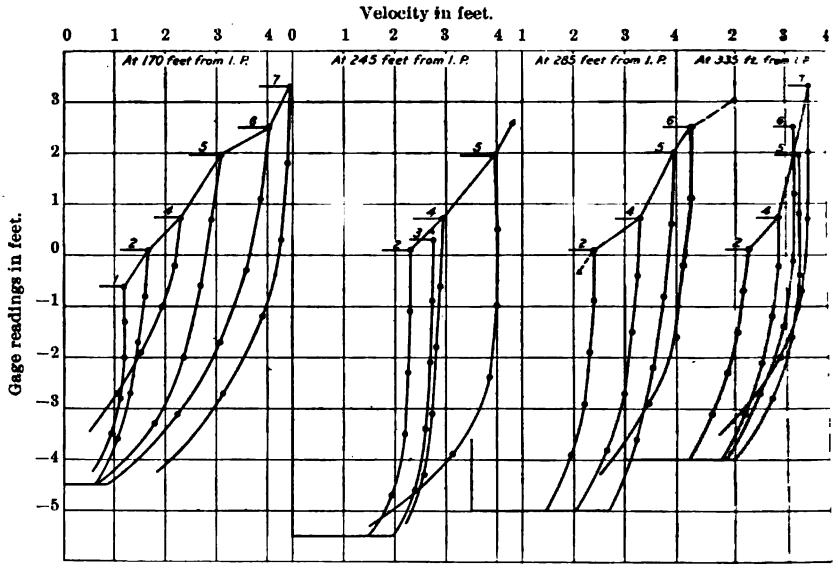

FIG. 51.—Vertical velocity curves, Susquehanna River, upper side of Exchange Street Bridge, Binghamton, N. Y.

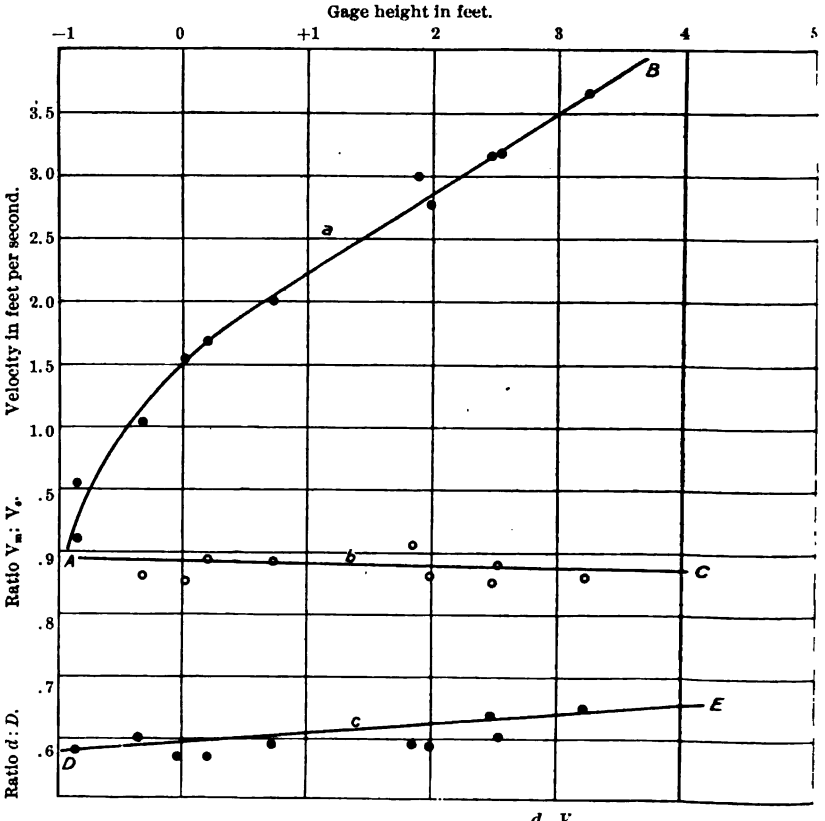

FIG. 52.—Diagrams showing change in velocity and in ratios $\frac{d}{D}$, $\frac{V_m}{V_s}$ for changes in river stage.

It is seen that the curves in fig. 50, out of the influence of the pier, are very regular in shape, and the surface velocities give a well-defined curve, the greatest velocity being at or near the surface.

Those in fig. 51 are not far from the piers, except that at 335 feet from initial point, which is near the shore. These are not so regular in shape, and the surface velocities do not fall on a well-defined curve. The points marked by triangles show surface velocity for the river stage indicated, but not taken when the vertical velocity curve was obtained.

Surface slope of Susquehanna River, Binghamton, N. Y., from Exchange Street Bridge to Washington Street Bridge, 1,850 feet.

Date.	Elevation water surface.		Difference elevation.
	Washington Street Bridge.	Exchange Street Bridge.	
1902.	*Feet.*	*Feet.*	*Feet.*
July 2	a 5.86	a 4.74	1.12
July 14	−6.95	−5.88	1.07
August 3	−4.44	−3.16	1.28
Do	−4.49	−3.16	1.33
Do	−4.53	−3.26	1.27
Do	−4.75	−3.42	1.33
August 4	−5.18	−3.91	1.27
Do	−5.45	−4.20	1.25
August 15	−7.33	−6.35	.98
Do	−7.41	−6.43	.98
Do	−7.40	−6.43	.97
Do	−7.37	−6.42	.96
Do	−7.39	−6.46	.93
August 16	−7.45	−6.55	.90
Do	−7.49	−6.62	.87

a Elevations referred to Binghamton City datum.

Changes in mean velocity in verticals with changes in river stage, upper side Exchange Street Bridge, Binghamton, N. Y.

Date.	Gage reading.	Distance of vertical from initial point.							
		60 feet.	100 feet.	125 feet.	170 feet.	245 feet.	285 feet.	310 feet.	335 feet.
1901.	*Feet.*								
July 10	−0.33	0.50	1.07	1.30	.....	.....	1.70	.....	.....
July 30	− .87	.....	.58	.....	.....	.....	.....	.....	.....
1902.									
Aug. 16	+ .04	.....	1.57	1.88	1.31	2.05	2.10	1.92	1.80
Aug. 15	+ .21	.93	1.66	2.03	1.36	2.22	2.26	2.03	2.01
July 14	+ .73	1.29	2.00	2.50	1.63	2.71	2.95	2.50	2.33
July 4	+1.85	1.89	3.00	3.46	2.26	.....	.....	.....	.....
July 2	+1.97	1.89	2.76	3.42	2.37	3.60	3.57	3.21	3.02
Aug. 4	+2.54	2.40	3.35	4.17	3.09	4.41	.....	3.31	2.88
Aug. 4	+2.46	2.35	3.32	4.09	3.09	.....	3.88	3.38	2.94
Aug. 3	+3.23	2.76	3.65	4.86	3.81	.....	.....	3.98	3.23

Changes in the ratio d : D in verticals for changes in river stage.

Gage reading.	Distance of vertical from initial point.							
	60 feet.	100 feet.	125 feet.	170 feet	245 feet	285 feet	310 feet.	335 feet.
Feet.								
−0.33	0.52	0.60	0.56	a 0.55	.....	0.57	.....	.....
− .87	.....	.58	.....	a .66	.....	.....	.....	.....
+ .04	.....	.57	.62	.56	0.69	.65	0.57	0.54
+ .21	.54	.57	.59	.58	.63	.61	.61	.57
+ .73	.58	.59	.62	.55	.63	.63	.60	.56
+1.85	.61	.59	.66	.64	.63	.....	.....	.....
+1.97	.50	.59	.67	.59	.69	.57	.56	.66
+2.54	.62	.60	.65	.63	.71	.....	.69	.64
+2.46	.65	.64	.66	.63	.....	.68	.68	.65
+3.23	.63	.65	.68	.61	.....	.....	.56	.61
Means..	.581	.598	.634	.600	.670	.620	.600	.600

a At 175 feet from initial point.

Changes in the ratio $V_m : V_s$ in verticals for changes in river stage.

Gage reading.	Distance of vertical from initial point.							
	60 feet.	100 feet.	125 feet.	170 feet.	245 feet.	285 feet.	310 feet.	335 fe
Feet.								
-0.33	0.89	0.86	0.90	[a] 0.87	.....	0.81	.....	.....
− .87	.....	.92	.....	[a] .89	.....	.....	.....	.....
+ .04	.....	.85	.86	.80	0.93	.91	0.84	0.
+ .21	.91	.88	.86	.81	.92	.89	.87	.8
+ .73	.90	.83	.86	.73	.93	.90	.88	.8
+1.85	.83	.91	.88	.75	.....	.....	.....	.....
+1.97	.79	.86	.89	.78	.90	.90	.89	.9
+2.54	.86	.88	.90	.76	.92	.....	.91	.9
+2.46	.88	.85	.89	.78	.....	.91	.92	.9
+3.23	.83	.86	.92	.86	.....	.....	.92	.9
Means..	.861	.870	.885	.808	.920	.887	.890	.8

[a] At 175 feet from initial point.

The slope of the surface from Exchange Street Bridge to Washington Street Bridge was obtained, for different river stages, by measuring from points of known elevation on these bridges. These data are given in the following table and plotted in fig. 53. It is seen that the slope increases more and more slowly as the surface elevation increases.

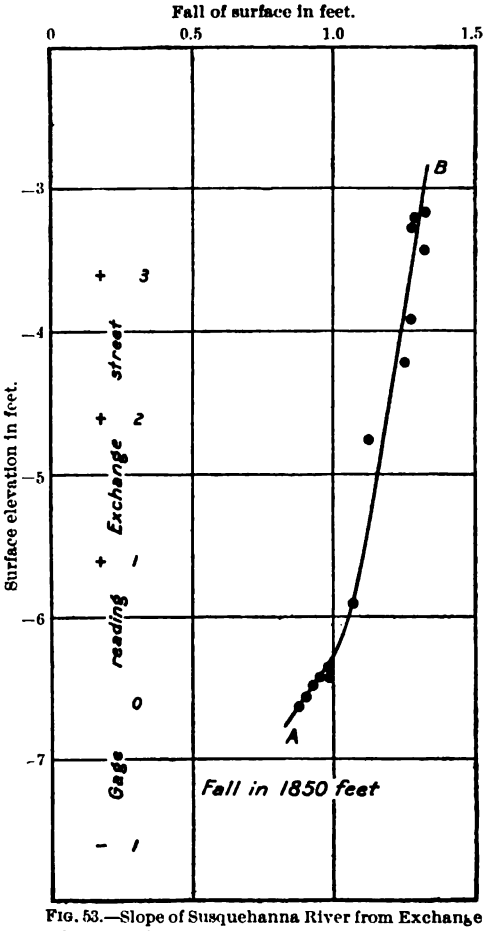

FIG. 53.—Slope of Susquehanna River from Exchange Street Bridge to Washington Street Bridge, Binghamton, N. Y.

The preceding table shows changes in mean velocity in the ratio $\frac{d}{D}$ and in the ratio $V_m : V_s$ in verticals for changes in river stage. The mean value of $\frac{d}{D}$ varies from 0.58 to 0.[illegible] and increases slightly with the depth for a given river stage and also increases somewhat as the stage increases. Fig. 52*c* illustrates this change in the vertical 100 feet from initial point. The mean value of the ratio $V_m : V_s$ varies from 0.86 to 0.92 (omitting that for 170 feet from initial point) and does not change much with change of stage. Fig. 52[illegible] illustrates this change for the vertical 100 feet. Fig. 52[illegible] illustrates the change in mean velocity with change in river stage. The velocity is seen to increase very rapidly as the stage increases.

WORK ON OSWEGO RIVER AT BATTLE ISLAND, NEW YORK.

Oswego River is formed by Oneida and Seneca rivers in the northern part of Onondaga County, N. Y., and flows northwest into Lake Ontario. It drains several of the lakes of central New York, and hence is not subject to the extreme changes of flow of many streams without natural storage. On the other hand, there are several dams on it where the water is held back to some extent and the flow rendered irregular. The Oswego and Syracuse canal takes water from it and to a slight extent increases the irregularity of the flow.

The gaging station is located 3 miles above Minetto, N. Y., and 8 miles from the river's mouth, just below a bend of large radius. Measurements are made from the car suspended from a cable (Pl. VI, *B*). The river bed is rock except near the shores, where it is gravel. The cross section is shown in fig. 54. The central part is triangular, and the station being at or just below a bend this shape modifies to a marked degree the vertical velocity curve near the upper edge of the east side. The bed rock is smooth, but is broken by irregular cracks, the edges of which are rough. The gage is of the wire-weight type, and is on the west bank, 2,900 feet upstream from the station. A temporary gage was put in at the cable, which, with the aid of the elevation of a bench mark at the

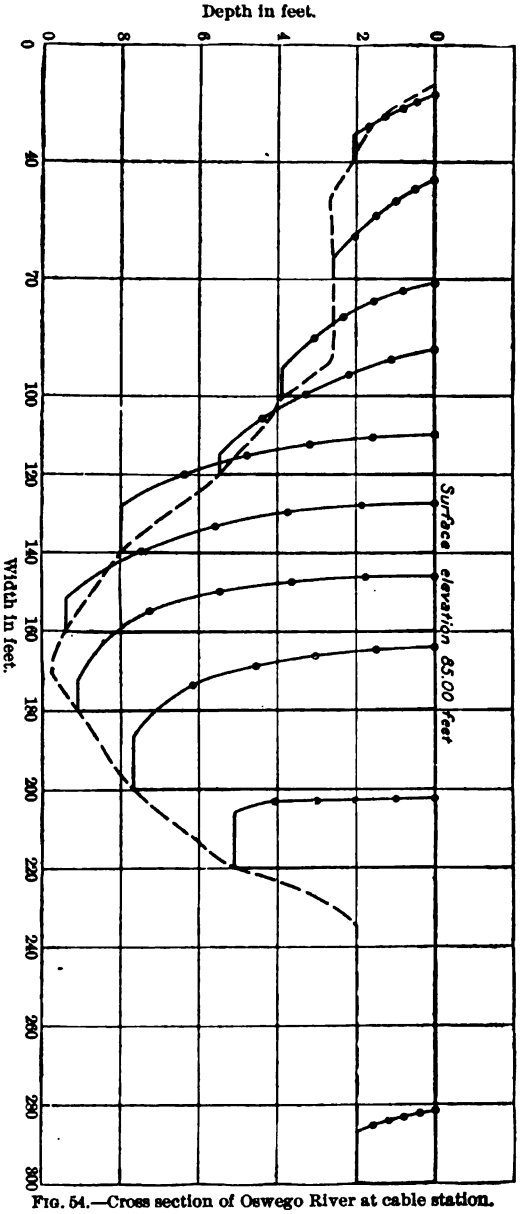

FIG. 54.—Cross section of Oswego River at cable station.

gage and one at the cable, enabled fluctuations in the surface elevation and the slope of the surface from the gage to the cable to be measured.

Fifteen discharge measurements were made here by three methods. The results of these are given in the following table:

Results of discharge measurements at Battle Island, New York, on Oswego River.

Date.	No.	Method.	Surface elevation cable.	Area.	Discharge.	Mean velocity.
1901.			*Feet.*	*Square feet.*	*Cubic feet.*	*Feet.*
July 13	1	0.6 depth	86.12	1,812	5,264	
Aug. 7	2	V. v. curves	85.04	1,344	3,224	
Aug. 7	3	0.6 depth	85.06	1,358	3,235	
Aug. 27	4	.....do	84.99	1,354	3,203	
Aug. 27	5	.....do	84.90	1,322	2,938	
Aug. 28	6	.....do	85.02	1,363	3,169	
Aug. 28	7	.....do	84.96	1,344	3,066	
1902.						
Mar. 31	8	Harlacher's	90.77	3,491	15,982	
Apr. 1	9	V. v. curves	90.63	3,440	15,653	
July 8	10	Harlacher's	87.85	2,421	8,601	
July 8	11	V. v. curves	87.83	2,414	8,634	
July 9	12	Harlacher's	87.88	2,431	9,019	
July 9	13	V. v. curves	87.85	2,421	8,565	
Aug. 19	14	Harlacher's	87.00	2,061	6,999	
Aug. 19	15	V. v. curves	86.88	2,018	6,562	

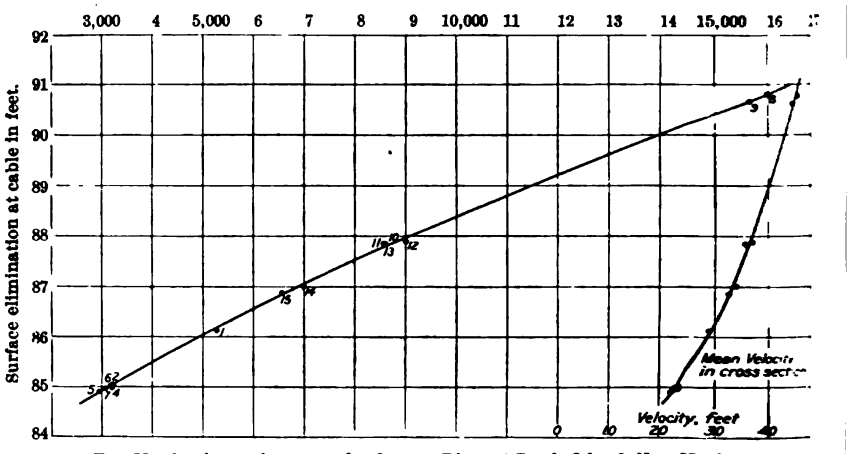

FIG. 55.—Station rating curve for Oswego River at Battle Island, New York.

These discharges and the corresponding surface elevations are plotted in fig. 55. From this curve the following station-rating table has been prepared:

Station-rating table for Oswego River, Battle Island, New York.

Surface elevation cable.	Discharge.	Surface elevation cable.	Discharge.	Surface elevation cable.	Discharge.
Feet.	*Cubic feet.*	*Feet.*	*Cubic feet.*	*Feet.*	*Cubic feet.*
85.0	3,130	87.0	6,850	89.0	11,380
.1	3,300	.1	7,060	.1	11,620
.2	3,480	.2	7,270	.2	11,870
.3	3,660	.3	7,480	.3	12,120
.4	3,840	.4	7,690	.4	12,370
.5	4,020	.5	7,910	.5	12,620
.6	4,200	.6	8,130	.6	12,870
.7	4,380	.7	8,350	.7	13,120
.8	4,560	.8	8,570	.8	13,370
.9	4,740	.9	8,790	.9	13,630
86.0	4,930	88.0	9,020	90.0	13,890
.1	5,120	.1	9,250	.1	14,150
.2	5,310	.2	9,480	.2	14,410
.3	5,500	.3	9,710	.3	14,670
.4	5,690	.4	9,940	.4	14,930
.5	5,880	.5	10,180	.5	15,200
.6	6,070	.6	10,420	.6	15,470
.7	6,260	.7	10,660	.7	15,740
.8	6,450	.8	10,900	.8	16,010
.9	6,650	.9	11,140	.9	16,280
				91.0	16,550

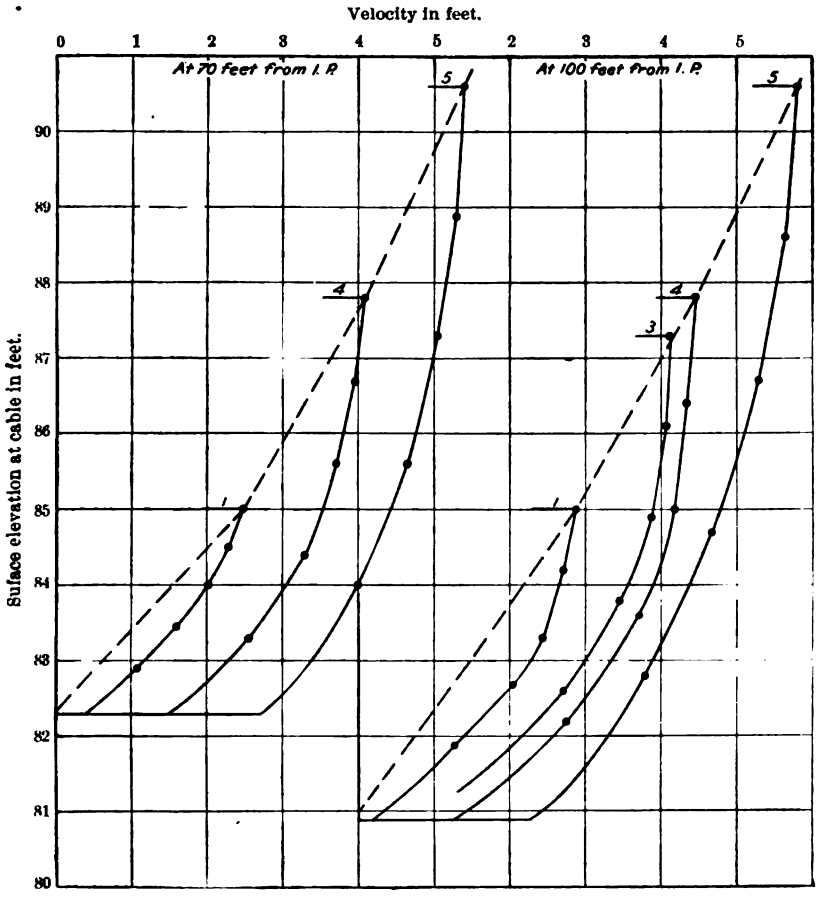

FIG. 56.—Vertical velocity curves for Oswego River at Battle Island, New York.

The following table prepared from the two preceding ones shows the percentage difference between the measured discharge and the mean discharge or that given by the station rating curve:

Comparison of measured and mean discharge, Oswego River, Battle Island, New York.

Date.	No.	Elevation, surface cable.	Discharge.		Discharge difference (M.–C.).	
			By measure (M.).	By curve (C.).		
1901.		*Feet.*	*Cubic feet.*	*Cubic feet.*	*Cubic feet.*	*Per cent.*
July 13	1	86.12	5,264	5,158	+106	+2.1
August 7	2	85.04	3,224	3,198	+ 26	+ .8
Do	3	85.06	3,235	3,232	+ 3	+ .1
August 27	4	84.99	3,203	3,113	+ 90	+2.9
Do	5	84.90	2,938	3,070	− 32	−1.0
August 28	6	85.02	3,169	3,164	+ 5	+ .2
Do	7	84.96	3,066	3,062	+ 4	+ .1
1902.						
March 31	8	90.77	16,368	15,929	+439	+2.8
April 1	9	90.63	15,653	15,551	+102	+ .7
July 8	10	87.85	8,610	8,680	− 70	− .8
Do	11	87.83	8,634	8,636	− 2	.0
July 9	12	87.88	9,019	8,746	+273	+3.1
Do	13	87.85	8,585	8,680	− 95	−1.1
July 19	14	87.00	6,999	6,850	+145	+2.1
Do	15	86.88	6,562	6,612	− 50	− .8

It is seen that the percentage difference, or variation of the measured from the mean discharge, is small, the largest being about 3 per cent. The four by Harlacher's method, Nos. 8, 10, 12, and 14, are too large by 2 to 3 per cent, showing that for this river, at this particular place, the coefficient in the formula $Q = c \Sigma V_s F$ should be 0.83 instead of 0.85. It was seen on pages 116, 125 that the extreme variation of the measured discharges from the mean discharge is from −8.3 to +5.1 per cent for the Chenango River station, from −5.3 to +2.1 per cent for the Susquehanna. It is from +2.89 to −1.09 per cent for Oswego River. The per cent variation being a measure of the accuracy, it is clear that the discharge measurements at the Oswego River station, where there are no obstructions, is more accurate by several per cent than those at the Chenango and Susquehanna river stations.

Nine sets of vertical velocity curves have been obtained in 12 verticals in substantially the same way as described on page 117. For the higher river stages the current was so swift in the deeper parts of the section that the meter was carried several feet downstream when the largest sinker that could be managed from the car was used. The position of the meter was obtained from the length of cable immersed, the angle it made with the vertical, and the distance of the car above the water.

The vertical velocity curves in each vertical are divided into five groups, and the mean of each group plotted in figs. 56, 57, and 58. From a study of these sets of vertical velocity curves, and those in fig. 54, it is seen that the greatest velocity is at or near the surface, and that

near the shore for shallow depths the vertical velocity curve is a nearly straight line. As the depth and velocity increase, this line becomes more curved and more inclined from the vertical, and the ratio $\frac{d}{D}$

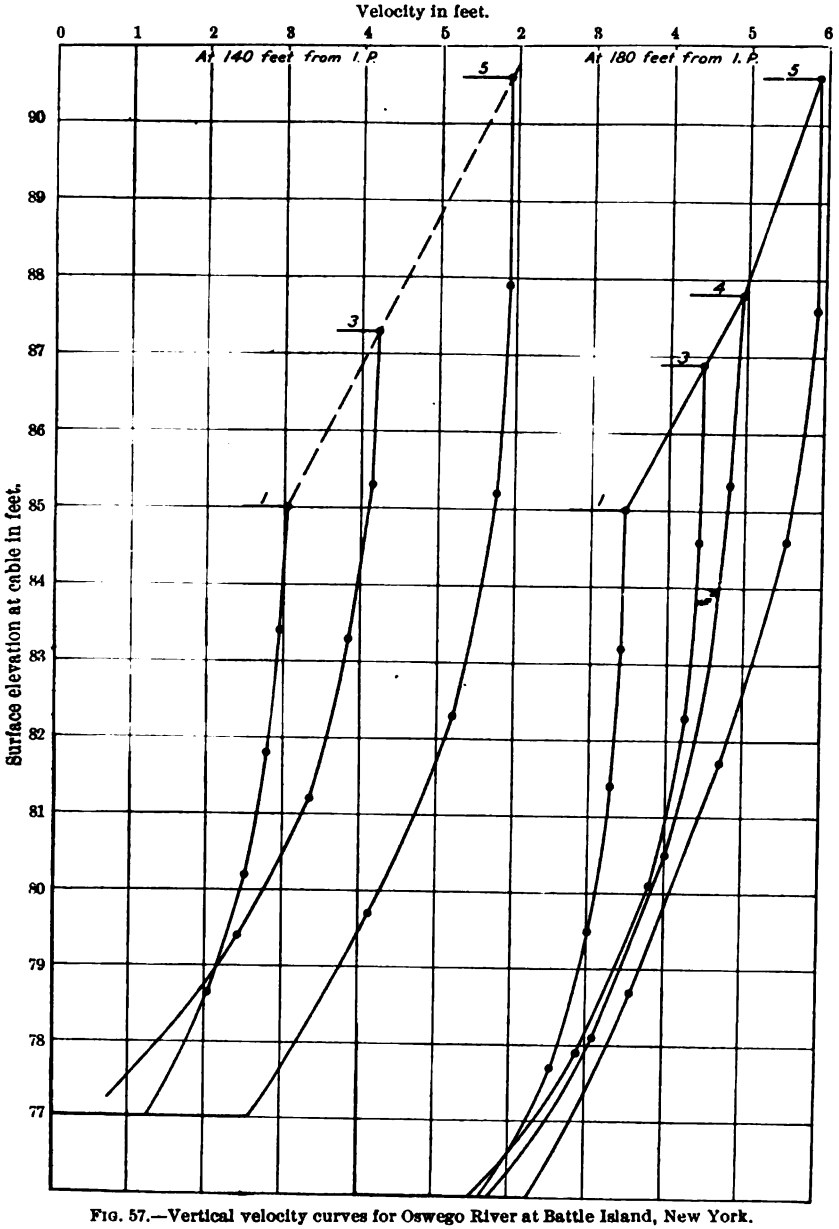

FIG. 57.—Vertical velocity curves for Oswego River at Battle Island, New York.

increases from 0.55 to 0.63. The effect of the bed and bend in the stream, is very noticeable in the curves at 220 feet from initial point in figs. 54 and 58. The stream lines striking the steep incline at this

point are thrown upward, causing "boiling" of the water and a decrease of velocity. As the river rises this phenomenon is less marked. The effect of the bend is further seen in the table on page 136. The ratio $\frac{V_m}{V_s}$ is much larger on the concave than on the convex part of the stream.

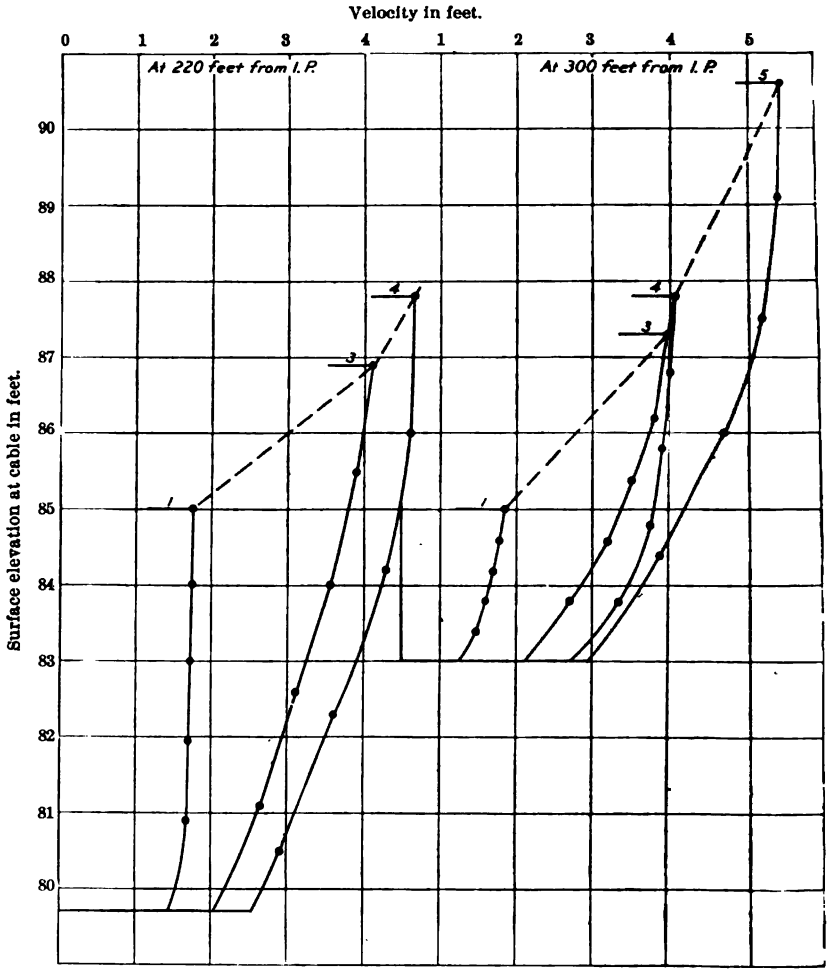

FIG. 58.—Vertical velocity curves for Oswego River at Battle Island, New York.

The shape of the bed has also a marked effect on the curves for different river stages at 180 feet.

The following table shows the change in mean velocity, the change in the ratio $\frac{d}{D}$, and the change in the ratio $\frac{V_m}{V_s}$ in verticals for changes in river stage. The mean velocity is seen to increase very rapidly with the increase in stage. The ratio of the depth of the thread of

mean velocity to the total depth increases in some verticals, in others it decreases; as a rule it increases slightly. The ratio of mean velocity

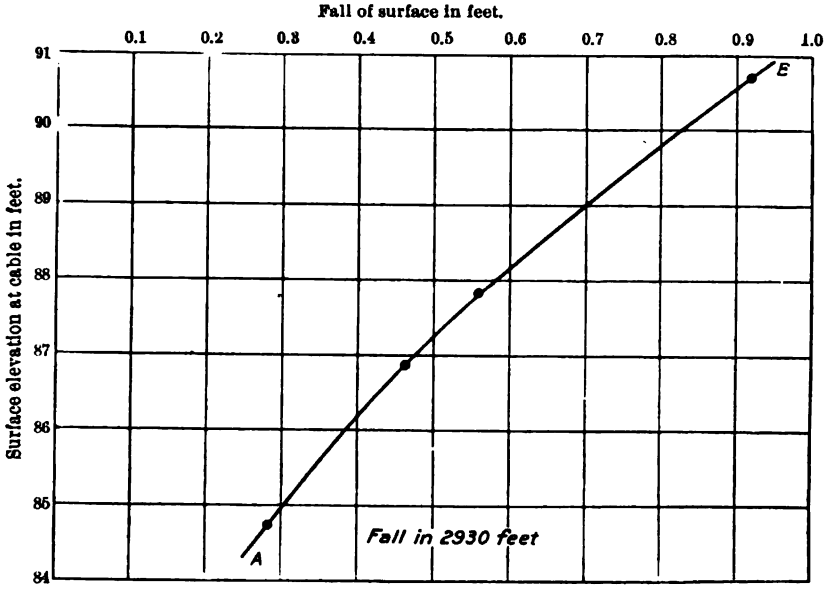

FIG. 59.—Slope of surface of Oswego River at Battle Island, New York.

to velocity 1 foot below the surface increases with the increase in stage, and is greater on the concave than on the convex side.

Data obtained on Oswego River at Battle Island, New York, showing change of mean velocity in verticals with change in river stages.

Date.	Surface elevation at cable.	Distances of verticals from initial point.											
		40.	70.	100.	120.	140.	160.	180.	200.	220.	240.	260.	300.
1901.	*Feet.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*
Aug. 27	84.95	1.19	1.79	2.10	2.12	2.43	2.58	2.92	3.01	1.63	1.98	.....	1.58
Aug. 28	85.00	.....	.....	2.20	2.14	2.51	2.66	2.97	3.12	1.70	.....	.....	.....
Aug. 7	85.04	1.28	1.71	2.08	2.32	2.57	2.74	3.01	3.34	1.76	2.12	2.19	1.67
July 13	86.07	1.73	2.60	.....	2.95	3.03	3.23	3.49	.....	2.82	.....	2.90	.....
1902.													
Aug. 19	86.88	2.22	3.10	3.50	3.34	.....	3.47	3.70	3.84	3.30	3.91	3.57	3.31
Aug. 19	86.95	.....	.....	3.37	3.29	3.30	3.55	3.65	3.92	3.22	.....	.....	.....
July 8	87.33	2.49	3.26	3.53	3.80	.....	3.93	3.94	.....	3.77	4.00	3.90	3.65
July 9	87.85	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Apr. 1	90.63	.....	4.59	4.73	.....	5.07	.....	4.79	.....	.....	.....	.....	.....
Depths	85.00	2.2	2.5	4.0	5.5	8.0	9.4	9.1	7.7	5.2	2.0	1.9	2.0

Table showing changes in the ratio d : D in verticals for changes in river stages.

Surface elevation at cable.	Distances of verticals from initial point.											
	40.	70.	100.	120.	140.	160.	180.	200.	220.	240.	260.	300.
Feet.	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*
84.95	0.54	0.53	0.58	0.60	0.61	0.61	0.64	0.59	0.60	0.50	0.50	0.50
85.00	.....	.....	.52	.56	.60	.65	.64	.60	.61	.....	.....	.....
85.04	.58	.60	.60	.56	.61	.64	.60	.60	.60	.55	.53	.53
86.07	.55	.56	.....	.63	.60	.64	.66	.....	.60	.....	.60	.....
86.88	.59	.60	.59	.61	.....	.61	.62	.66	.55	.55	.64	.53
86.95	.....	.....	.62	.61	.67	.65	.67	.66	.61	.56	.54	.....
87.83	.70	.55	.65	.66	.....	.61	.62	.....	.55	.57	.....	.63
87.85	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.66	.....
90.63	.....	.59	.59	.....	.63	.....	.60	.60	.....	.58	.....	.58
Means.	.582	.572	.592	.602	.620	.630	.631	.618	.567	.552	.580	.554

Table showing changes in the ratio $V_m : V_s$ in verticals for changes in river stages.

Surface elevation at cable.	Distances of verticals from initial point.											
	40.	70.	100.	120.	140.	160.	180.	200.	220.	240.	260.	300.
Feet.	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*	*Feet per sec.*
84.95	0.79	0.77	0.79	0.71	0.80	0.82	0.87	0.85	0.99	0.92	0.92	0.93
85.00	.....	.....	.76	.70	.83	.82	.87	.87	.96	.....	.....	.....
85.04	.88	.75	.71	.71	.86	.81	.85	.91	.98	.92	.92	.93
86.07	.83	.75	.....	.77	.80	.80	.85	.....	.80	.....	.82	.....
86.88	.85	.85	.83	.77	.80	.77	.82	.84	.83	.93	.94	.86
86.95	.....	.....	.82	.77	.79	.81	.83	.87	.78	.....	.....	.....
87.83	.85	.82	.86	.84	.....	.82	.82	.....	.81	.91	.94	.92
87.85	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
90.63	.....	.85	.82	.....	.86	.....	.81	.89	.....	.85	.....	.87
Means.	.840	.798	.799	.753	.820	.807	.840	.872	.879	.906	.908	.902

The surface slope data obtained at this station are given in the following table:

Slope of surface of Oswego River at Battle Island, New York, in a distance of 2,930 feet, for variation of river stage.

Date.	Elevation surface.		*Eg – Ec.*	Mean.
	At gage.	At cable.		
1901.	*Feet.*	*Feet.*	*Foot.*	*Foot.*
August 26	[a] 85.16	[a] 84.87	0.29	
August 27	84.98	84.71	.27	
Do	85.03	84.77	.26	0.278
Do	85.03	84.75	.28	
August 28	84.93	84.66	.27	
Do	85.08	84.78	.30	
1902.				
March 31	91.71	90.77	.94	
Do	91.68	90.75	.93	.92
April 1	91.51	90.62	.89	
July 8	88.44	87.85	.59	
Do	88.42	87.83	.59	.557
July 9	88.41	87.88	.53	
Do	88.37	87.85	.52	
August 19	87.45	86.99	.46	
Do	87.34	86.88	.46	.463
Do	87.35	86.88	.47	

[a] The elevations of bench marks at gage and cable were furnished by R. F. Horton.

These data have been combined as shown, giving the fall in the surface in 2,930 feet for four river stages. These are plotted in fig. 59, giving the curve AB. This curve is seen to be concave to the horizontal axis, whereas the slope curves for Chenango and Susquehanna rivers, figs. 46 and 53, are convex to this axis.

CONCLUSIONS.[a]

In the following statements relative to methods and accuracy of current meter work done on a stream at a gaging station it is assumed that the channel is approximately straight, regular in shape, with few obstructions, and the bed is such that there are few projections of more than 4 inches in height above its general contour. It is also assumed that there is a permanent gage at the station that can be read to hundredths of a foot and that is frequently checked by measurement from a permanent bench mark.

(1) The distance of the thread of mean velocity below the surface increases with the depth and with the ratio of depth to width. In general it varies from about 0.55 to 0.65 of the depth.

(2) In a broad, shallow stream from 3 to 12 inches in depth, and having a sand or fine gravel bed, the thread of mean velocity is from 0.50 to 0.55 depth below the surface. The mean velocity in a vertical is the half sum of the velocities found by holding the center of the meter 0.15 foot below the surface and then 0.15 foot above the bed. If the bed is coarse gravel (particles 1 to 2½ inches in diameter) the center of the meter should be held 0.15 foot below the surface and then from 0.3 to 0.4 foot above the bed.

(3) In broad streams from 1 to 3 feet in depth, and having gravelly beds, the thread of mean velocity is from 0.55 to 0.60 of the depth below the surface. The single-point method of measuring the velocity, holding the center of the meter 0.58 of the depth below the surface, will give good results.

(4) In ordinary streams where the depth varies from about 1 to 6 feet, the thread of mean velocity is about 0.6 below the surface.

(5) In the smaller streams (creeks) having a width of from 20 to 40 feet, the thread of mean velocity is farther below the surface than in a broader stream of the same depth.

(6) When the velocity is more than 2 feet per second the center of the Price meter should not be held nearer to the surface than 1 foot.

(7) The top and bottom method (center of meter half a foot below the surface and then half a foot above the bed of the stream) gives velocity from 2 to 8 per cent or more greater than the mean velocity. This method should be used only where the bed is comparatively smooth and visible.

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[a] See also pp. 98, 150, 151, and 160.

(8) The vertical integration method is rapid, and is a very good method to use as a check on the single-point method. It gives better results than the single-point method at a bend in a stream, under ice, under logs, or where the conditions are poor.

(9) The single-point method gives satisfactory results where the conditions are good, but the hydrographer must use his judgment as to the depth of the thread of mean velocity below the surface, which may be from 0.55 to 0.65 of the depth.

(10) The vertical-velocity-curve method of obtaining mean velocity is the most accurate, but requires the longest time to operate it. A few vertical velocity curves should be taken at each measurement at characteristic places in the cross section as a check on the results obtained by the other method.

(11) At stations with fairly permanent bed and without obstructions, the individual discharge measurements when plotted should not vary from the station rating curve by more than 3 to 5 per cent for ordinary river stages. For very low stages it may be 10 per cent, and for high water, where the banks overflow, the variation from the mean rating curve may be 20 per cent.

(12) At stations with permanent bed and channel obstructions the variation of a plotted discharge from the station rating curve may be 8 per cent of the discharge for ordinary stages.

(13) A discharge measurement should never be considered complete until some test has been applied to see if it satisfies some criterion of accuracy, as those in (11) or (12) or some other, and if it does not satisfy the proper criterion the reason should be sought in the change in channel, in gage, or in mistakes.

(14) For flood measurements and in cases where the velocity is so great that the meter can not be kept at about 0.6 depth, the velocity 1 foot below the surface should be obtained and the discharge computed from the formula $Q = C(V'_s F' + V''_s F'' + \ldots)$ where V'_s, V''_s, ... are the velocities 1 foot below the surface; F', F'', ... are the component areas; and C a coefficient whose value varies from 0.95 for deep streams to 0.85 for streams of somewhat less than moderate size and depth. An easily recognized mark on the meter cable will facilitate the keeping of the meter 1 foot below the surface.

(15) Discharge should not be measured with a current meter in a section where the mean velocity is less than half a foot per second in more than 15 per cent of it. Usually during low water a section can be found within a reasonable distance of the station with a velocity of 1 foot or more and with a comparatively smooth bed, to which access can be had by wading.

VERTICAL-VELOCITY-CURVE MEASUREMENTS.

ON SOUTHERN RIVERS.

During 1903 a series of 301 vertical-velocity-curve measurements were made on a number of rivers in Virginia, North Carolina, South Carolina, and Tennessee, by E. W. Myers and Brent S. Drane, of the Survey. In the majority of instances these observations were made after a gaging of the stream by the six-tenths depth method, the object being to get, for that gaging, the coefficient for reducing the velocities observed at six-tenths depth to mean velocity. At times, however, the entire gaging would be made by taking the vertical velocity curved at regular intervals of five or ten feet, across the section. In these instances the velocity at six-tenths depth was usually not observed.

The top-and-bottom method was not used. The velocities tabulated as observed at surface were really observed at a depth of 0.5 foot below the surface, Mr. E. C. Murphy having found in his Cornell experiments that the small Price meter does not register the true velocity when run just beneath the surface, if the velocity is greater than about 2 feet per second. Similarly, no readings were taken closer than 0.5 foot from the bottom, this being the distance of the center of the meter wheel above the bottom of the weight.

With only two exceptions the measurements were made from bridges. Properly, the meter should be lowered from the downstream side of the bridge, and have an additional cable attached to it, running down to it from the upstream side of the bridge, thus insuring its being lowered vertically in the section. This practice was followed at only one station, at Judson, N. C., on Little Tennessee River. At all other stations the meter was swung downstream more or less by the current, and the velocities were not strictly in the vertical plane of the section; the slant was so slight, however, as not to be of material consequence.

In measuring, the meter was carefully lowered until the middle of the meter wheel was just on the water surface. It was then lowered 0.5 foot and run for fifty seconds. This reading having been recorded, it was lowered 0.5 foot, placing it 1 foot below the surface. The meter was run here, and in like manner downward in succession, at intervals of an amount dependent on the depth and on the time at the disposal of the hydrographer, the last reading being taken with the meter weight resting on the stream bottom. The meter was then raised until the bottom of the weight just touched the water surface, and the distance through which it had been raised was carefully measured. This was the slant depth, and was recorded in such a manner as to avoid confusion with the vertical sounding previously made to develop the section.

The field notes of a typical curve are as follows:

[Gaging made January 23, 1904, by B. S. Drane, meter No. 349, on Dan River, at Madison, State of North Carolina. Gage height: beginning 2.10 feet, ending 2.26 feet, mean 2.18 feet. River rising.]

Soundings.		Observations.		Total number of revolutions.	Revolutions per second.	Velocity per second.	Per cent of (slant) depth.
Distance from initial point.	Depth.	Depth of observations.	Time in seconds.				
460	a3.0	0.5	50	60	1.20	2.88	17
		1.0	50	58	1.16	2.78	33
		1.5	50	54	1.08	2.59	50
		2.0	50	47	.94	2.27	67
		2.5	50	38	.76	1.83	83

a Equal slant.

In computing results, the first step was to express the depths at which velocities were observed, as decimal parts of the slant depth. This operation was very quickly performed by the help of a slide rule, and the decimal fractions entered in the field notebook, each immediately after its velocity. This is indicated as having been done in the field notes above.

The observations were then plotted on cross-section paper, record book 9–209 being very convenient for the purpose. Depths were plotted as ordinates and velocities as abscissas. A convenient scale is 4 inches to the total (slant) depth, this being considered as unity, and 1 inch for each foot per second of velocity.

A smooth curve was next drawn through the points thus plotted, care being taken to give them, as nearly as possible, equal weight, if all did not fall on a smooth curve. The abscissas were read off from the drawing, at the surface at each tenth of the distance from top to bottom and at the bottom, and set down in order. Thus, from the above curve were set down:

.s—2.90	.6—2.40
.1—2.90	.7—2.19
.2—2.87	.8—1.93
.3—2.81	.9—1.64
.4—2.71	b—1.31
.5—2.58	

The mean abscissa, or mean velocity, was then computed from these by an expanded form of the prismoidal formula for eleven abscissæ, as follows:

$$V_m = \tfrac{1}{30}[4(V_{.1}+V_{.3}+V_{.5}+V_{.7}+V_{.9})+2(V_{.2}+V_{.4}+V_{.6}+V_{.8})+V_s+V]$$

Combining the above abscissas according to this formula, gave—

.1— 2.90	.2— 2.87
.3— 2.81	.4— 2.71
.5— 2.58	.6— 2.40
.7— 2.19	.8— 1.93
.9— 1.64	
12.12	9.91
4	2
48.48	
19.82	19.82
a— 2.90	
b— 1.31	
30) 72.51	
2.417=V_m.	

The mean velocity for this curve was thus 2.417 feet per second, a velocity found to lie on the curve at 0.59 of the depth from the surface; this was marked on the curve in the cross-section book. The velocities $V_{0.6D}$, V_{Top}, and $\frac{1}{2}(V_{Top}+V_{Bottom})$ were read off from the curve, and the coefficients $\frac{V_m}{V_{0.6D}}$, $\frac{V_m}{V_{Top}}$, and $\frac{V_m}{V_{\frac{1}{2}(T+B)}}$ computed.

A notebook was kept for these computations in which all data were preserved. Above each set of curves was entered the river and station, the date, and the gage height, with the number of the current-meter notebook in which the field notes were to be found, and the page of the cross-section book on which the curves were drawn. Similar cross references were entered in the original field notes and in the cross-section book, thus making it easy to collect quickly all the information and computation of any particular curve.

In the following tables the general character of the bottom at each point of observation is indicated, as follows: M, mud or soft bottom; S, hard sand bottom; G, gravel bottom; R, smooth rock bottom; B, rough bottom of bowlders.

Vertical velocity measurements in 1903 on rivers in Virginia, North Carolina, South Carolina, and Tennessee.

APPOMATTOX RIVER,[a] MATTOAX, VA.

Date.	Gage height.	Distance from initial point.	Depth.	Velocity in feet per second by following methods: Vertical velocities.	Observed 0.6 depth.	0.6 depth from curve.	Observed top and bottom.	Observed top.	Coefficient for reducing to mean velocity. 0.6 depth from curve.	Observed top and bottom.	Observed top.	Depth of thread of mean velocity. In feet.	In per cent of depth.	Character of bottom.
July 25	1.50	100	3.3	2.215	2.260	2.220	2.490	2.720	1.00	0.89	0.81	2.01	61.0	S.
		110	3.3	2.295	2.190	2.290	2.040	2.390	1.00	1.12	.96	1.97	59.6	S.
		115	3.2	2.214	2.190	2.250	2.160	2.330	.98	1.02	.95	2.06	64.5	S.
May 21	2.33	90	2.5	.920	.810	.820	.950	1.690	1.12	.97	.54	1.32	53.0	S.
		100	4.1	2.020	2.070	2.070	1.980	2.340	.98	1.02	.86	2.58	63.0	S.
		110	4.5	2.150	2.370	2.320	1.920	2.610	.93	1.12	.82	3.15	70.0	S.
		130	4.1	1.760	1.760	1.760	1.380	2.640	1.00	1.27	.67	2.46	60.0	S.
Aug. 29	4.62	90	4.9	2.238	2.330	2.260	1.500	2.590	.99	1.49	.86	2.99	61.0	S.
Mean.									1.00	1.11	0.81		61.5	

JAMES RIVER,[b] CARTERSVILLE, VA.

May 22	2.42	90	4.4	1.880	.....	1.840	1.760	2.130	1.02	1.07	0.88	2.53	57.6	S.
		200	3.0	2.200	.....	2.220	1.660	2.470	1.00	1.32	.89	1.83	61.0	S.
		350	3.8	2.270	2.370	2.300	2.200	2.580	.99	1.03	.88	2.36	62.0	S.
		500	4.0	1.960	2.200	1.930	1.900	2.370	1.02	1.03	.83	2.32	58.0	S.
		640	3.8	2.370	.....	2.355	2.340	2.580	1.01	1.01	.92	2.18	57.5	S.
		760	4.0	2.340	2.300	2.350	2.240	2.580	1.00	1.04	.91	2.46	61.5	S.
Mean.									1.01	1.08	.88		59.6	

ROANOKE RIVER,[c] ROANOKE, VA.

July 20	1.03	95	3.3	0.871	0.810	0.880	0.800	0.840	0.99	1.09	0.93	2.05	62.0	S.
May 13	1.40	25	2.4	1.485	1.530	1.570	1.660	2.030	.95	.89	.73	1.49	62.0	S.
		40	2.9	1.730	1.570	1.730	1.300	2.030	.89	1.33	.85	1.87	64.5	S.
		50	2.9	1.346	1.500	1.275	1.560	2.030	1.06	.86	.66	1.62	56.0	S.
		55	1.8	1.960	1.820	1.990	2.020	2.130	.98	.97	.92	1.16	64.5	S.
		60	3.0	1.500	1.410	1.450	1.460	2.000	1.03	1.03	.75	1.74	58.0	S.
		90	2.5	1.720	1.860	1.760	1.700	1.830	.98	1.01	.99	1.60	64.0	R.
		95	1.8	1.310	1.120	1.270	1.510	1.930	1.03	.87	.68	1.01	56.0	R.
Mean.									.99	1.01	.81		60.9	

STAUNTON RIVER,[d] RANDOLPH, VA.

Aug. 28	3.61	200	4.2	2.394	2.300	2.450	2.130	2.330	0.98	1.12	1.03	2.67	63.5	S.
		230	2.8	2.726	2.990	2.800	2.590	3.050	.97	1.05	.89	1.76	63.0	S.
		260	2.0	2.810	2.920	2.840	2.770	3.090	.99	1.01	.91	1.22	61.0	S.
		330	2.3	1.350	1.600	1.300	1.430	1.800	1.04	.94	.75	1.25	54.5	S.
		360	3.7	2.417	2.630	2.460	2.290	2.520	.98	1.06	.96	2.29	62.0	G.
		390	2.0	2.920	2.920	2.980	2.850	2.850	.98	1.02	1.02	1.34	67.0	G.

[a] This station is on the steel deck bridge of the Southern Railway, 27 miles south of Richmond, Va., at Mattoax station. The right bank is high and steep, but the left bank is low, and overflowed by floods. The bottom is firm, of sand and gravel. The current is steady at low water and ordinary stages, but very irregular in flood stages.

[b] The station here is on the wooden wagon bridge between Cartersville post-office and Pemberton station, on the James River branch of the Chesapeake and Ohio Railroad. There are 6 spans, with a total width of 790 feet at low water. The bottom is remarkably uniform, of sand, gravel, and small rocks, and the current very regular.

[c] The old station, where these curves were made, is at the Walnut Street Bridge, on the edge of the city of Roanoke. The bottom is of rock and sand, very uneven, and the current irregular and shifting.

[d] This station is on the steel bridge of the Richmond and Danville branch of the Southern Railway, near Randolph, Va. At ordinary stages the stream flows under two spans. Floods spread out under all four spans and overflow the left bank; there are two flood channels through the long embankment on which the track approaches the bridge, on left bank. The bottom is soft, of silt and fine sand along left bank; but becomes firmer toward right bank, of sand and gravel. The current is steady and of fairly good velocity at all stages.

Vertical velocity measurements in 1908 on rivers in Virginia, etc.—Continued.

STAUNTON RIVER, RANDOLPH, VA.—Continued.

Date.	Gage height.	Distance from initial point.	Depth.	Velocity in feet per second by following methods: Vertical velocities.	Observed 0.6 depth.	0.6 depth from curve.	Observed top and bottom.	Observed top.	Coefficient for reducing to mean velocity. 0.6 depth from curve.	Observed top and bottom.	Observed top.	Depth of thread of mean velocity. In feet.	In per cent of depth.	Character of bottom.
Aug. 8	4.60	200	4.8	2.504	2.390	2.540	2.260	2.330	.99	1.11	1.07	3.10	64.5	S.
		220	4.3	2.849	2.990	2.870	2.660	2.920	.99	1.07	.98	2.64	61.5	S.
		240	3.7	3.111	3.120	3.210	2.780	3.320	.97	1.14	.94	2.48	67.0	S.
		340	4.7	2.802	3.380	2.920	2.250	2.990	.96	1.25	.94	3.08	65.5	S.
		370	3.5	1.952	1.730	2.060	1.700	1.930	.95	1.15	1.01	2.31	66.0	G.
		390	3.8	2.778	3.290	2.900	1.870	2.990	.96	1.49	.93	2.47	65.0	G.
		420	4.4	2.103	2.560	2.250	1.830	1.930	.93	1.15	1.09	3.43	78.0	G.
June 24	5.28	200	5.2	2.290	2.660	2.370	2.210	2.520	.97	1.04	.91	3.38	65.0	S.
		220	5.2	2.758	2.660	2.810	2.510	2.890	.98	1.10	.95	3.35	64.5	S.
		240	4.5	2.725	3.320	2.920	2.350	3.320	.93	1.16	.82	2.99	66.5	S.
		260	4.1	2.900	3.110	3.110	2.610	3.370	.93	1.11	.86	2.75	67.0	S.
		340	6.0	2.822	3.230	3.070	2.270	2.890	.92	1.24	.98	4.44	74.0	S.
		380	5.0	2.750	2.990	2.970	2.410	2.680	.93	1.14	1.03	3.55	71.0	G.
		400	6.0	2.692	2.990	2.860	2.460	3.110	.94	1.09	.87	4.05	67.5	G.
Mean.									.96	1.12	.95		65.7	

DAN RIVER,[a] MADISON, N. C.

Oct. 1	0.86	510	1.9	0.734	0.783	0.730	0.751	0.880	1.00	0.98	0.83	1.14	60.0	M.
		520	3.8	1.688	1.570	1.620	1.640	1.880	1.01	1.00	.87	2.20	58.0	M.
		560	1.7	1.777	1.830	1.730	1.780	2.220	1.04	1.00	.80	.99	58.0	M.
July 18	1.14	455	2.7	2.791	.....	2.880	2.670	2.820	.97	1.05	.99	1.71	63.5	G.
		460	2.7	2.641	2.790	2.690	2.400	2.790	.98	1.10	.95	1.67	62.0	G.
		500	3.0	2.133	2.130	2.120	2.100	2.430	1.01	1.02	.88	1.77	59.0	M.
		550	2.3	2.312	.....	2.540	1.510	2.660	.91	1.53	.87	1.53	66.5	M.
June 18	2.05	450	2.5	2.308	.....	2.450	2.240	2.940	.94	1.03	.79	1.60	64.0	G.
		455	2.7	2.680	.....	3.000	2.770	3.180	.89	.97	.84	1.86	69.0	G.
		460	3.2	2.741	.....	2.810	2.630	3.130	.98	1.04	.88	2.08	65.0	G.
		465	3.5	2.948	.....	3.050	2.800	3.080	.97	1.05	.96	2.33	66.5	G.
		470	3.4	2.987	.....	3.010	2.890	3.300	.99	1.03	.91	2.07	61.0	G.
		475	3.5	2.996	.....	3.000	2.900	3.470	1.00	1.03	.86	2.10	60.0	G.
		480	3.1	2.881	.....	2.970	2.800	3.320	.97	1.03	.87	1.97	63.5	G.
		485	2.8	2.645	.....	2.680	2.630	3.080	.99	1.01	.86	1.71	61.0	G.
		490	3.1	2.664	.....	2.680	2.610	3.080	.99	1.02	.86	1.89	61.0	G.
		495	3.3	2.642	.....	2.650	2.620	2.990	1.00	1.01	.88	2.00	60.5	S.
		500	3.3	2.592	.....	2.570	2.560	2.940	1.01	1.01	.88	1.96	59.0	M.
		505	3.1	2.528	.....	2.650	2.470	2.990	.95	1.02	.85	2.06	66.0	M.
		510	3.2	2.556	.....	2.680	2.400	3.130	.95	1.06	.82	2.08	65.0	M.
		515	3.2	2.410	.....	2.440	2.440	3.030	1.00	1.00	.81	1.92	60.0	M.
		520	3.7	1.720	.....	1.570	2.210	2.560	1.10	.78	.67	1.96	53.0	M.
		545	2.8	1.480	.....	1.420	1.820	2.180	1.04	.81	.68	1.62	58.0	M.
		550	2.5	2.019	.....	2.000	2.070	2.370	1.01	.98	.85	1.47	59.0	M.
		555	1.8	2.100	.....	2.100	2.190	2.370	1.00	.96	.89	1.08	60.0	M.
		560	1.4	2.086	.....	2.120	2.180	2.280	.98	.97	.91	.98	64.0	M.
		565	1.7	1.806	.....	1.840	1.900	2.130	.98	.95	.85	1.05	61.5	M.
		570	1.6	1.933	.....	1.960	2.020	2.230	.99	.96	.87	.98	61.5	M.
		575	1.8	1.490	.....	1.450	1.570	2.040	1.03	.95	.73	1.05	58.5	M.
		580	1.3	1.290	.....	1.390	1.540	1.570	.93	.84	.82	.86	66.5	M.
		585	1.4	1.600	.....	1.750	1.590	1.470	.91	1.01	1.09	.98	70.0	M.
Dec. 21	2.05	515	3.2	1.859	.....	1.700	2.000	2.590	1.09	.93	.72	1.68	52.5	S.
		555	3.0	2.314	.....	2.420	2.140	2.710	.96	1.08	.85	1.90	63.5	M.
		565	2.6	2.541	.....	2.500	2.610	2.810	1.02	.97	.90	1.38	53.0	M.
		575	2.6	2.479	.....	2.510	2.430	2.780	.99	1.02	.89	1.61	62.0	M.
May 14	2.07	450	2.4	2.180	.....	2.180	2.140	2.610	1.00	1.02	.84	1.44	60.0	G.
		460	3.1	2.540	.....	2.620	2.430	2.900	.97	1.05	.88	2.00	64.4	G.
		470	3.1	2.800	.....	2.830	2.830	3.090	.99	.99	.91	1.92	62.0	G.
		490	2.7	2.750	.....	2.780	2.680	3.020	.99	1.03	.91	1.69	62.5	G.
		510	3.7	2.800	.....	2.830	2.670	2.880	.99	1.05	.97	2.36	63.7	S.
		550	3.0	2.600	.....	2.610	2.630	2.950	.99	.99	.88	1.86	62.0	M.
		570	2.4	2.466	.....	2.470	2.460	2.730	1.00	1.00	.83	1.44	60.0	M.
Mean.									.99	1.01	.86		61.6	

[a] This station is on the covered wooden bridge of the Southern Railway, about one-half mile from Madison. At all ordinary stages the river flows under only two spans. The banks are of loose sand, and the bottom to right of central pier is of coarse, loose sand, subject to change. Under the span to the left of central pier the bottom is hard sand, except for a short section close to the pier. The current is uniform and fairly swift.

Vertical velocity measurements in 1903 on rivers in Virginia, etc.—Continued.

DAN RIVER,[a] SOUTH BOSTON, VA.

Date.	Gage height.	Initial distance from point.	Depth.	Velocity in feet per second by following methods: Vertical velocities.	Observed 0.6 depth.	0.6 depth from curve.	Observed top and bottom.	Observed top.	Coefficient for reducing to mean velocity. 0.6 depth from curve.	Observed top and bottom.	Observed top.	Depth of thread of mean velocity. In feet.	In per cent of depth.	Character of bottom.
Aug. 27	1.40	160	5.6	1.508	1.570	1.600	1.390	1.670	0.94	1.08	0.90	3.75	67.0	S.
		180	5.5	1.953	1.930	1.940	1.880	2.260	1.01	1.04	.86	3.25	59.0	S.
		280	4.7	1.968	1.910	1.970	1.900	2.130	1.00	1.04	.92	2.87	61.0	S.
		323	3.5	.952	.....	.960	.910	1.010	.99	1.05	.94	2.17	62.0	S.
Aug. 8	2.62	275	6.1	2.399	.....	2.440	2.170	2.520	.98	1.11	.95	3.81	62.5	S.
		300	5.7	2.021	2.330	2.060	1.900	2.330	.98	1.06	.87	3.59	63.0	S.
		320	5.3	1.883	1.800	1.970	1.640	1.800	.96	1.15	1.05	3.76	71.0	S.
June 23	3.71	130	9.0	.900	.660	.900	.780	1.140	1.00	1.15	.79	5.40	60.0	M.
		150	8.8	2.065	2.250	2.260	1.850	2.280	.91	1.12	.91	5.85	69.0	R.
		180	7.9	2.295	2.420	2.360	2.070	2.660	.97	1.11	.86	5.02	63.5	S.
		260	8.1	2.413	2.710	2.500	2.070	2.850	.97	1.11	.85	5.31	65.5	S.
		300	6.7	2.215	2.280	2.250	2.040	2.610	.98	1.09	.85	4.19	62.5	S.
		330	5.1	1.510	1.640	1.520	1.380	1.590	.99	1.09	.95	3.09	60.5	S.
Mean.	.....	.....	.....	.....	.....	.....	.....	.....	.98	1.09	.90	.....	63.6	

REDDIE RIVER,[b] NORTH WILKESBORO, N. C.

July 17	[c]24.67	45	1.7	1.620	1.600	1.570	1.730	2.060	1.03	0.94	0.79	0.96	56.5	S.
		55	2.1	2.010	.....	2.000	2.040	2.300	1.00	.99	.87	1.24	59.0	S.
		70	2.5	1.640	1.590	1.610	1.670	2.000	1.02	.98	.82	1.44	57.5	S.
Mean.	.....	.....	.....	.....	.....	.....	.....	.....	1.02	.97	.83	.....	57.7	

YADKIN RIVER,[d] NORTH WILKESBORO, N. C.

Oct. 3	0.38	80	3.5	1.203	1.160	1.200	1.210	1.310	1.00	0.99	0.92	2.10	60.0	M.
		90	4.1	1.372	1.350	1.370	1.330	1.550	1.00	1.03	.89	2.46	60.0	M.
		110	3.4	2.092	2.070	2.070	1.960	2.360	1.01	1.07	.89	2.01	59.0	M.
		120	3.0	2.242	2.640	2.250	2.120	2.640	1.00	1.06	.85	1.82	60.5	S.
		130	2.7	2.648	2.190	2.690	2.280	2.190	.98	1.16	1.21	1.70	63.0	S.
		140	3.0	2.583	2.740	2.560	2.460	3.880	1.01	1.05	.90	1.77	59.0	S.
Nov. 6	0.71	80	3.8	2.397	2.570	2.550	2.220	2.310	.97	1.08	1.04	2.77	73.0	M.
		100	5.2	2.556	2.760	2.700	2.370	2.460	.95	1.08	1.04	3.85	74.0	M.
		130	3.2	2.063	2.220	2.100	1.930	2.070	.98	1.07	1.00	2.02	63.0	S.
July 16	1.15	70	4.2	1.514	1.860	1.500	1.160	1.970	1.01	1.31	.77	2.48	59.0	M.
		80	4.5	2.390	2.540	2.400	2.250	3.320	1.00	1.06	.72	2.72	60.5	M.
		90	5.3	2.680	.....	2.850	2.320	2.790	.94	1.16	.96	3.55	67.0	M.
		100	5.4	2.967	3.050	3.050	2.860	3.050	.97	1.04	.97	4.05	75.0	M.
		110	5.3	2.800	3.050	3.040	2.070	2.990	.92	1.35	.94	3.60	68.0	S.
		120	4.3	2.960	2.920	2.990	2.760	3.180	.99	1.07	.93	2.62	61.0	S.
		130	3.3	2.530	3.050	2.780	2.330	2.920	.90	1.09	.87	2.21	67.0	S.
June 17	1.46	70	4.3	1.787	.....	1.710	1.810	2.420	1.04	.99	.74	2.45	57.0	M.
		75	4.4	2.845	.....	3.050	2.590	3.180	.93	1.10	.89	3.01	68.5	M.
		85	5.5	2.821	.....	3.050	2.450	3.080	.92	1.15	.92	4.18	76.0	M.
		95	6.0	2.909	.....	3.040	2.630	3.180	.96	1.11	.91	4.50	75.0	M.
		105	6.3	2.927	.....	3.050	2.300	2.750	.93	1.27	1.06	4.91	78.0	S.
		115	6.3	3.162	3.320	3.310	2.880	2.990	.96	1.10	1.06	4.95	78.5	S.
		125	4.5	3.120	3.230	3.240	3.250	3.230	.96	.96	.97	3.37	75.0	S.
		135	3.8	2.876	2.760	2.810	2.940	3.270	1.02	.98	.88	2.00	52.5	S.
		145	4.2	2.737	3.130	3.070	2.260	3.370	.89	1.21	.81	2.90	69.0	S.

[a] This station is on the steel bridge of the Norfolk and Western Railway. At ordinary stages there is only one obstruction in the section—the tubular steel central pier. Floods do not spread far on left bank, but overflow a wide flood plain on right bank. A portion of a dam washed down from above, stranded near left bank about 200 feet above the bridge, makes the current uneven for about 50 feet from left bank. The bottom in lee of this obstruction is of soft mud. Elsewhere it is hard, clayey sand.

[b] There is no regular station on this stream, which is a tributary of Yadkin River. The bottom is smooth and hard, of sand.

[c] Below bench mark.

[d] This station is on the lower wagon bridge, crossing the river about 1 mile from Wilkesboro railroad station. At ordinary stages there is no obstruction in the section, but floods overflow the right bank around a stone pier and trestle beats. The current makes a slight angle with the bridge, and there is a pulsation in it. The bottom is somewhat shifting, of coarse sand, with some bowlders close to left bank.

Vertical velocity measurements in 1903 on rivers in Virginia, etc.—Continued.

YADKIN RIVER, NORTH WILKESBORO, N. C.—Continued.

Date.	Gage height.	Distance from initial point.	Depth.	Velocity in feet per second by following methods: Vertical velocities.	Observed 0.6 depth.	0.6 depth from curve.	Observed top and bottom.	Observed top.	Coefficient for reducing to mean velocity. 0.6 depth from curve.	Observed top and bottom.	Observed top.	Depth of thread of mean velocity. In feet.	In per cent of depth.	Character of bottom.
June 16	1.56	70	4.4	2.150	.....	2.080	1.870	2.980	1.05	1.15	0.73	2.46	56.0	M.
		80	5.2	3.026	3.320	3.160	2.780	3.280	.96	1.09	.94	3.69	71.0	M.
		90	6.0	2.767	.....	3.040	2.190	3.320	.91	1.26	.83	4.41	73.5	M.
		100	6.4	3.070	.....	3.330	2.680	3.270	.92	1.15	.94	4.58	71.5	M.
		110	5.9	3.026	.....	3.350	2.300	3.490	.90	1.32	.87	4.19	71.0	S.
		120	5.8	2.648	.....	2.700	2.570	3.180	.98	1.03	.83	3.62	62.5	S.
		130	4.5	2.335	.....	2.580	1.900	2.850	.90	1.23	.82	3.06	68.0	S.
		140	4.3	2.990	.....	3.080	2.870	3.270	.97	1.04	.91	2.75	64.0	S.
Mean.	.....	.....	.....	.....	.....	.....	.....	.....	.96	1.12	.91	.....	66.5	

YADKIN RIVER,[a] SALISBURY, N. C.

Dec. 17	1.80	120	4.0	0.780	0.690	0.760	0.720	1.020	1.08	1.08	0.76	2.32	58.0	S.
		140	3.7	.958	.950	.900	.960	1.090	1.01	1.00	.88	2.18	59.0	S.
		160	6.1	1.063	1.190	1.060	1.020	1.350	1.01	1.04	.79	3.66	60.0	S.
June 20	2.57	80	3.1	1.970	2.000	2.090	1.920	2.180	.94	1.03	.90	2.05	66.0	S.
		110	3.2	1.700	1.230	1.630	1.700	2.280	1.04	1.00	.75	1.78	57.5	S.
		200	3.1	2.540	2.800	2.610	2.470	2.760	.97	1.03	.92	2.02	65.0	S.
		260	4.3	1.990	1.850	1.830	1.870	2.890	1.09	1.06	.69	2.37	55.0	S.
		360	5.4	2.950	2.850	2.970	2.370	3.320	.99	1.24	.89	3.32	61.5	S.
		380	11.3	3.280	4.180	2.980	3.040	4.650	1.08	1.06	.69	6.10	54.0	S.
		420	3.9	2.980	3.130	2.880	2.900	3.660	1.03	1.03	.81	2.16	55.5	S.
Mean.	.....	.....	.....	.....	.....	.....	.....	.....	1.02	1.06	.81	.....	59.2	

CATAWBA RIVER,[b] MORGANTON, N. C.

Oct. 28	1.16	50	1.9	1.072	1.040	1.050	1.150	1.310	1.02	0.92	0.82	1.10	58.0	S.
		70	3.0	1.029	.980	.970	1.100	1.470	1.06	.91	.70	1.68	56.0	S.
		90	3.5	1.367	1.260	1.330	1.300	1.690	1.03	1.05	.81	2.03	58.0	S.
		110	3.2	1.420	1.300	1.390	1.330	1.740	1.02	1.07	.82	1.86	58.0	S.
		130	2.8	1.636	1.720	1.600	1.600	1.980	1.02	1.02	.83	1.61	57.5	G.
		150	3.5	1.190	1.130	1.180	1.160	1.450	1.01	1.03	.82	2.08	59.5	G.
		170	3.5	.825	.830	.825	.800	.970	1.00	1.03	.85	2.10	60.0	G.
Dec. 11	1.20	50	1.8	.851	.690	.810	.910	1.190	1.05	.94	.72	1.03	57.5	R.
		60	2.8	.858	.990	.900	.830	1.070	.96	1.03	.80	1.85	66.0	R.
		70	3.0	1.002	.870	1.040	.980	1.310	.96	1.02	.76	1.89	63.0	R.
		80	3.3	1.105	1.190	1.100	1.100	1.450	1.00	1.00	.76	1.98	60.0	R.
		90	3.3	1.293	1.280	1.290	1.220	1.690	1.00	1.06	.77	1.98	60.0	R.
		100	3.2	1.351	1.480	1.330	1.360	1.790	1.02	.99	.75	1.86	58.0	S.
		110	3.2	1.370	1.160	1.310	1.370	1.930	1.05	1.00	.71	1.79	56.0	S.
		120	3.0	1.685	1.620	1.730	1.600	1.980	.97	1.05	.85	1.89	63.0	S.
		130	3.3	1.651	1.690	1.680	1.570	1.980	.98	1.05	.86	2.06	62.5	S.
		140	3.2	1.489	1.350	1.430	1.450	1.830	1.04	1.03	.81	1.74	54.5	S.
		140	3.2	1.394	1.470	1.400	1.360	1.640	1.00	1.02	.85	1.92	60.0	S.
		150	3.1	1.193	1.140	1.190	1.190	1.350	1.00	1.00	.88	1.86	60.0	S.
		160	3.4	.786	.830	.830	1.740	1.020	.95	1.06	.77	2.23	65.5	S.
		160	3.5	.799	.870	.820	.770	1.070	.97	1.04	.75	2.17	62.0	S.
		170	3.6	.742	.790	.770	.740	.920	.96	1.00	.81	2.27	63.0	S.
		180	3.1	.799	.740	.760	.740	.830	.98	1.08	.96	1.92	62.0	S.

[a] All of the following curves were made at the Piedmont Fall Bridge, several hundred yards above the railroad bridge, where the gage is placed, about 6 miles from Salisbury, N. C. The railroad bridge is preferable for gagings in time of highest floods, which cut around the end of the tall bridge on left bank, but the bottom is rocky and rough, and the current is whirled in all directions by ragged rock ledges just above. For this reason the toll bridge is to be preferred for gaging all ordinary stages. At the toll bridge the current is regular at all stages. The bottom is sand, except under first span from left bank, where it is rock.

[b] This station is on the Lenoir wagon bridge, 1 mile from Morganton, a one-span steel bridge, with wood trestle approach on right bank. The left bank is a high rock cliff; the right bank is sand, sloping gradually up above flood height. Along the left bank the bottom is rock for a short distance, then of coarse sand and gravel, which changes very little. The current is very even and well distributed across.

Vertical velocity measurements in 1903 on rivers in Virginia, etc.—Continued.

CATAWBA RIVER, MORGANTON, N. C.—Continued.

Date.	Gage height.	Distance from initial point.	Depth.	Velocity in feet per second by following methods: Vertical velocities.	Observed 0.6 depth.	0.6 depth from curve.	Observed top and bottom.	Observed top.	Coefficient for reducing to mean velocity. 0.6 depth from curve.	Observed top and bottom.	Observed top.	Depth of thread of mean velocity. In feet.	In per cent of depth.	Character of bottom.
Aug. 21	1.76	70	3.8	1.716	1.440	1.740	1.460	2.460	0.99	1.18	0.70	2.34	61.5	R.
		120	4.3	2.432	2.390	2.440	2.230	2.520	1.00	1.09	.93	2.64	61.5	S.
		170	3.5	1.874	2.160	1.900	1.780	2.060	.99	1.05	.91	2.17	62.0	S.
June 25	1.96	70	4.0	1.861	1.950	1.870	1.540	2.080	1.00	1.21	.89	2.42	60.5	R.
		80	3.9	2.233	1.930	2.220	2.160	2.800	1.01	1.03	.80	2.32	59.5	R.
		100	4.2	2.845	2.950	2.860	2.830	3.130	.99	1.01	.91	2.62	62.5	S.
		135	5.0	2.120	2.400	2.160	1.830	2.610	.98	1.16	.81	3.10	62.0	S.
		160	4.0	2.048	1.950	2.070	1.900	2.230	.99	1.08	.92	2.50	62.5	S.
		180	3.5	1.861	1.830	1.940	1.730	2.040	.96	1.08	.91	2.31	66.0	S.
May 5	2.00	50	3.0	2.043	2.410	2.000	2.000	2.510	1.02	1.02	.81	1.72	57.5	R.
		60	3.8	2.223	2.470	2.310	1.860	2.440	.96	1.20	.91	2.41	63.5	R.
		70	5.0	1.636	1.960	1.520	1.690	2.370	1.08	.97	.69	2.70	54.0	R.
		80	3.7	2.465	1.830	2.450	2.440	3.120	1.00	1.01	.79	2.20	59.5	R.
		90	3.9	2.605	2.580	2.620	2.820	3.050	.99	.92	.85	2.36	60.5	R.
		125	5.0	2.623	2.710	2.760	2.440	2.980	.95	1.08	.88	3.45	69.0	S.
		150	5.0	2.601	2.780	2.620	2.490	2.810	.99	1.04	.93	3.02	60.5	S.
Mean.									1.00	1.04	.82		60.6	

CATAWBA RIVER, MOUNT HOLLY, N. C. (MISCELLANEOUS); BOTTOM MUDDY; CURRENT SLUGGISH.

Nov. 13	(a)	80	6.3	0.494	0.450	0.470	0.480	0.610	1.05	1.03	0.81	3.34	53.0	M.
	(a)	180	7.0	.743	.720	.770	.660	.780	.96	1.13	.95	4.48	64.0	M.
Mean.									1.00	1.08	.88		58.5	

CATAWBA RIVER, BELMONT, N. C. (MISCELLANEOUS); BOTTOM HARD SAND.

Dec. 3	(a)	110	3.5	1.538	1.620	1.520	1.460	1.930	1.01	1.05	0.80	2.06	59.0	S.
Mean.									1.01	1.05	.80		59.0	

WATEREE (CATAWBA) RIVER,[b] CAMDEN, S. C. ON TOLL BRIDGE, 3 MILES FROM CAMDEN.

May 26	8.35	180	12.3	1.430	1.330	1.440	1.340	1.620	1.00	1.07	0.88	7.38	60.0	M.
		280	14.2	1.330	1.370	1.270	1.090	1.500	1.04	1.22	.89	7.53	53.0	M.
June 23	10.80	180	16.4	1.780	1.830	1.770	1.590	1.970	1.01	1.12	.90	9.51	58.0	M.
		280	17.7	1.835	1.830	1.850	1.670	1.930	.99	1.10	.95	11.06	62.5	M.
Mean.									1.01	1.13	.90		58.4	

BROAD RIVER,[c] ALSTON, S. C.

June 24	6.30	70	6.1	2.786	2.850	2.830	2.630	2.990	0.98	1.06	0.93	3.87	63.5	M.
		200	6.2	4.000	.....	4.210	2.920	4.500	.95	1.37	.89	4.15	67.0	S.
		350	8.9	3.479	.....	3.630	2.670	4.310	.96	1.30	.81	5.61	63.0	S.
		480	5.0	3.929	.....	4.190	3.440	4.240	.94	1.14	.93	3.40	68.0	S.
Mean.									.96	1.22	.89		65.4	

[a] No gage.
[b] The section is deep, with mud bottom, and the current very sluggish.
[c] This station is on the bridge of the Southern Railway, at Alston, S. C. The current is steady, the bottom of fine sand, occasionally muddy, and subject to slight change.

Vertical velocity measurements in 1903 on rivers in Virginia, etc.—Continued.

SALUDA RIVER,[a] WATERLOO, S. C.

Date.	Gage height.	Distance from initial point.	Depth.	Velocity in feet per second by following methods:					Coefficient for reducing to mean velocity.			Depth of thread of mean velocity.		Character of bottom.
				Vertical velocities.	Observed 0.6 depth.	0.6 depth from curve.	Observed top and bottom.	Observed top.	0.6 depth from curve.	Observed top and bottom.	Observed top.	In feet.	In per cent of depth.	
June 25	7.15	640	8.5	2.007	.....	2.040	1.770	2.330	0.98	1.13	0.86	5.31	62.5	S.
		670	7.1	2.242	.....	2.260	2.190	2.510	.99	1.02	.89	4.33	61.0	S.
		750	3.0	1.761	.....	1.860	1.830	2.510	.95	.95	.70	1.89	63.0	S.
Mean.	.....	.....	.....	.....	.....	.....	.....	.....	.97	1.03	.82	.....	62.2	

WATAUGA RIVER,[b] ELIZABETHTON, TENN.

Sept. 5	1.25	55	4.1	1.012	.....	1.010	1.020	1.160	1.00	0.99	0.87	2.46	60.0	R.
Nov. 26	1.26	30	3.2	.344	0.384	.380	.322	.384	.90	1.07	.90	2.18	68.0	S.
		40	3.3	.422	.480	.470	.380	.500	.90	1.11	.84	2.24	68.0	S.
		65	5.3	.461	.465	.450	.440	.600	1.02	1.05	.77	3.05	57.5	S.
July 3	1.61	40	3.7	.266	.....	.290	.210	.280	.92	1.27	.95	2.40	65.0	S.
		45	3.5	.439	.....	.420	.420	.650	1.05	1.05	.68	2.40	57.0	S.
		55	4.3	.401	.....	.390	.410	.610	1.03	.98	.66	2.54	59.0	S.
		155	3.0	.692	.....	.660	.730	.870	1.05	.95	.80	1.57	52.5	S.
		195	3.0	.702	.....	.740	.690	.780	.95	1.02	.90	2.01	67.0	S.
		210	2.6	.643	.....	.650	.530	.790	.99	1.21	.81	1.56	60.0	G.
Mean.	.....	.....	.....	.....	.....	.....	.....	.....	.98	1.07	.82	.....	61.4	

SOUTH FORK, HOLSTON RIVER, BLUFF CITY, TENN.

Sept. 4	0.11	210	3.1	1.084	.....	1.080	1.090	1.260	1.00	0.99	0.86	1.86	60.0	R.
July 2	.40	200	1.9	2.628	.....	2.540	2.920	3.910	1.35	.90	.67	1.10	58.0	B.
		205	2.4	5.066	.....	5.010	5.010	6.230	1.01	1.01	.81	1.39	58.0	B.
		215	3.6	2.695	.....	2.880	2.530	2.590	.94	1.07	1.04	2.77	77.0	B.
		250	2.8	3.366	.....	3.360	3.150	3.840	1.00	1.07	.88	1.68	60.0	B.
		255	2.3	2.346	.....	2.500	1.880	2.520	.94	1.25	.93	1.52	66.0	B.
May 9	1.50	115	2.7	6.770	0.890	.770	.740	1.220	1.00	1.04	.63	1.62	60.0	B.
		120	2.2	1.540	.....	1.600	1.560	1.760	.96	.99	.88	1.43	65.0	B.
		130	2.8	1.650	.....	1.680	1.630	2.200	.98	1.01	.75	1.71	61.0	B.
		140	3.7	1.770	.....	1.830	1.590	2.340	.97	1.11	.76	4.60	63.0	B.
		210	3.9	4.860	.....	4.520	4.980	6.800	1.08	.98	.70	2.07	53.0	B.
		240	4.1	2.290	.....	2.220	2.020	3.600	1.03	1.13	.64	2.38	58.0	B.
		250	4.1	4.210	.....	4.310	4.020	5.090	.98	1.05	.83	2.54	62.0	B.
Mean.	.....	.....	.....	.....	.....	.....	.....	.....	1.02	1.05	.80	.....	61.6	

NOLICHUCKY RIVER,[c] GREENEVILLE, TENN.

Oct. 15	0.00	100	1.6	0.935	0.870	0.900	1.010	1.210	1.04	0.93	0.77	0.90	56.0	B.
		120	1.6	.775	.720	.730	.840	1.160	1.06	.82	.67	.90	57.0	B.
		140	1.8	.903	.970	.900	.900	1.020	1.00	1.00	.39	1.08	60.0	B.
		170	2.3	.487	.450	.480	.540	.690	1.02	.90	.71	1.06	59.0	B.
		230	3.8	.627	.610	.640	.610	.760	1.00	1.03	.82	2.36	62.0	B.
		260	3.7	.743	.770	.750	.710	.970	.99	1.05	.77	2.28	61.5	B.
July 1	0.61	100	2.2	1.663	.....	1.650	1.640	2.000	1.01	1.01	.83	1.30	59.0	B.
		120	2.4	1.536	1.400	1.510	1.540	2.000	1.02	1.00	.77	1.42	59.0	B.
		140	2.5	1.416	1.340	1.400	1.400	1.730	1.00	1.01	.82	1.46	58.5	B.

[a] This station is on the bridge of the Charleston and Western Carolina Railroad, 10 miles north of Greenwood, S. C., and is best reached by driving from Greenwood. The current is regular; the bottom sandy and somewhat shifting.

[b] This station is on the bridge of the Virginia and Southwestern Railway, at Siam station, 3 miles from Elizabethton, Tenn. The bottom is of fine sand and gravel, with a shoal of small rocks in mid stream. The current is small at low water.

[c] This station is at Jones's Bridge, a two-span steel wagon bridge, 6 miles from Greeneville. The banks are steep and high; the left bank is never overflowed, and the right only by the highest floods. The current is even and fairly swift. The bottom is of smooth rock and small rounded stones.

Vertical velocity measurements in 1903 on rivers in Virginia, etc.—Continued.

NOLICHUCKY RIVER, GREENEVILLE, TENN.—Continued.

Date.	Gage height.	Distance from initial point.	Depth.	Velocity in feet per second by following methods:					Coefficient for reducing to mean velocity.			Depth of thread of mean velocity.		Character of bottom.
				Vertical velocities.	Observed 0.6 depth.	0.6 depth from curve.	Observed top and bottom.	Observed top.	0.6 depth from curve.	Observed top and bottom.	Observed top.	In feet.	In per cent of depth.	
July 1	0.61	160	2.5	2.143	1.070	1.110	1.160	1.370	1.17	1.12	0.82	1.45	58.0	B.
		220	4.4	1.826	.....	.850	.670	1.200	.98	1.24	.69	2.73	62.0	B.
		250	4.3	1.138	.....	1.240	.970	1.400	.92	1.17	.81	2.92	68.0	B.
May 7	1.48	80	2.9	1.620	.....	1.610	1.610	1.790	1.01	1.01	.92	1.71	59.0	B.
		100	3.1	2.680	2.640	2.720	2.640	3.150	.98	1.02	.85	1.89	61.0	B.
		120	3.2	2.690	2.510	2.720	2.770	3.190	.99	.97	.84	1.95	61.0	B.
		140	3.3	2.550	2.640	2.640	2.570	2.850	.97	.99	.89	2.18	66.0	B.
		160	3.4	2.160	2.100	2.100	2.100	2.580	1.03	1.03	.84	1.94	57.0	B
		180	4.5	1.650	.....	1.520	1.670	2.240	1.09	.99	.74	2.43	54.0	B.
		220	4.7	1.700	.....	1.520	1.770	2.370	1.11	.96	.72	2.35	50.0	B.
		240	5.1	2.200	.....	2.110	2.100	2.640	1.04	1.05	.83	2.86	56.0	B.
		270	4.6	2.460	.....	2.460	2.170	2.850	1.00	1.13	.86	2.76	60.0	B.
		290	4.9	2.070	.....	1.970	2.000	2.780	1.05	1.04	.74	2.74	56.0	B.
Mean.	.....	.....	.....	.....	.....	.....	.....	.....	1.02	1.02	.80	.....	59.1	

FRENCH BROAD RIVER,[a] OLDTOWN (NEWPORT), TENN.

Oct. 14	1.31	270	7.3	0.894	.....	0.890	0.900	1.110	1.00	0.99	0.81	4.34	59.5	S.
		300	5.3	.787	.....	.780	.760	.950	1.01	1.04	.83	3.13	59.0	S.
		410	5.4	.742	.....	.750	.690	.860	.99	1.08	.86	3.27	60.5	S.
		440	4.2	.527	0.550	.520	.570	.690	1.01	.92	.76	2.50	59.5	S.
June 30	2.29	40	5.3	1.233	1.340	1.240	1.110	1.340	.99	1.11	.92	3.19	60.2	S.
		140	5.3	2.116	2.260	2.190	1.920	2.360	.97	1.10	.90	3.52	66.5	S.
		205	6.2	2.178	.....	2.170	2.060	2.590	1.00	1.06	.84	3.69	59.5	S.
		280	8.0	1.933	.....	2.000	1.570	1.930	.97	1.23	1.00	5.20	65.0	S.
		420	6.0	1.842	.....	1.880	1.740	2.130	.98	1.06	.86	3.87	64.5	S.
Mean.	.....	.....	.....	.....	.....	.....	.....	.....	.99	1.07	.86	.....	61.6	

PIGEON RIVER,[b] NEWPORT, TENN.

Nov. 20	1.10	90	4.4	0.827	0.810	0.820	0.800	0.950	1.01	1.03	0.87	2.77	63.0	S.
June 30	1.51	30	6.9	.823	.....	.860	.730	.860	.96	1.13	.96	4.31	62.5	M
		40	11.6	.847	.780	.850	.650	.910	1.00	1.30	.93	7.02	60.5	M
		60	10.1	1.142	.940	1.130	.990	1.320	1.01	1.15	.87	5.96	59.0	M
Mean.	.....	.....	.....	.....	.....	.....	.....	.....	.99	1.15	.91	.....	61.2	

TUCKASEGEE RIVER,[c] BRYSON, N. C.

Oct. 9	1.16	30	5.7	0.496	.....	0.510	0.560	0.740	0.97	0.89	0.67	3.53	62.0	M.
		40	5.6	.547	.....	.600	.520	.500	.91	1.05	1.09	3.92	70.0	M.
		60	4.6	.833	.....	.830	.830	1.020	1.00	1.00	.82	2.76	60.0	M.
June 26	1.70	20	3.5	.673	0.690	.690	.670	.870	.98	1.00	.77	2.20	63.0	M.
June 27		25	5.1	.763	.....	.770	.810	1.000	.99	.94	.76	3.22	62.0	M.
		30	6.0	1.093	1.230	1.190	.990	1.100	.92	1.10	.99	4.32	72.0	M.
		35	6.3	1.231	.....	1.270	1.170	1.350	.97	1.05	.91	4.47	71.0	M.
		40	6.0	1.211	1.230	1.300	1.050	1.050	.93	1.15	1.15	4.53	75.5	M.
		45	6.1	1.039	.....	1.150	.850	.960	.90	1.22	1.09	4.58	75.0	M.
		50	6.3	1.010	.....	1.020	.960	1.100	.99	1.05	.92	3.94	62.5	M.
		55	6.4	1.198	.....	1.186	1.170	1.330	1.00	1.02	.90	4.00	62.5	M.

[a] This station is on the Oldtown steel wagon bridge, 3 miles from Newport, Tenn. The banks are high and not overflowed. The current is regular. The bottom is of fine sand and shifts slightly.

[b] This station is on the wagon bridge, 1 mile east of Newport. The section is bad, being deep and shallow water above and below. The bottom is mud, changing constantly. The current is sluggish.

[c] This station is on the highway bridge in the town of Bryson. The bottom is soft, and changes slightly. But for one old pier, the section is very smooth. The current is of fair velocity, except near right bank, where it is quite sluggish at low water. There is a decided pulse in current.

Vertical velocity measurements in 1903 on rivers in Virginia, etc.—Continued.

TUCKASEGEE RIVER, BRYSON, N. C.—Continued.

Date.	Gage height.	Distance from initial point.	Depth.	Velocity in feet per second by following methods: Vertical velocities.	Observed 0.6 depth.	0.6 depth from curve.	Observed top and bottom.	Observed top.	Coefficient for reducing to mean velocity. 0.6 depth from curve.	Observed top and bottom.	Observed top.	Depth of thread of mean velocity. In feet.	In per cent of depth.	Character of bottom.
June 27		60	5.8	1.591	1.710	1.720	1.380	1.660	0.92	1.15	0.96	4.20	72.5	M.
		65	5.6	1.611	1.710	1.710	1.430	1.760	.94	1.13	.92	3.95	70.5	M.
		70	5.3	1.684	.....	1.660	1.620	2.040	1.01	1.04	.83	3.07	58.0	M.
		75	4.9	1.836	.....	1.820	1.770	2.080	1.01	1.04	.88	2.82	57.5	M.
		80	4.3	1.902	1.880	1.940	1.750	2.020	.98	1.09	.94	2.80	65.0	M.
		85	4.0	1.670	1.520	1.650	1.690	2.040	1.01	.99	.82	2.34	58.5	M.
		90	4.0	1.828	1.810	1.830	1.780	2.130	1.00	1.03	.86	2.40	60.0	M.
		95	4.6	1.645	.....	1.640	1.660	1.850	1.00	.99	.89	2.40	60.0	M.
		100	4.5	1.873	.....	1.890	1.780	2.320	.99	1.05	.81	2.79	62.0	M.
		105	4.6	1.850	.....	1.850	1.910	2.000	1.00	.97	.92	2.76	60.0	M.
		110	4.9	2.044	.....	2.040	1.970	2.280	1.00	1.04	.90	2.94	60.0	M.
		115	5.5	1.743	2.000	1.810	1.590	1.960	.96	1.10	.89	3.58	65.0	M.
		120	6.0	1.826	.....	1.860	1.570	2.080	.98	1.16	.88	3.12	62.5	M.
		130	5.5	1.678	.....	1.640	1.570	2.230	1.02	1.07	.75	3.14	57.0	M.
Apr. 25	2.70	30	6.8	2.040	2.020	2.100	1.750	1.970	.97	1.17	1.04	1.70 4.28	25.0 63.0	M.
		40	6.9	1.970	.....	1.970	1.790	1.970	1.00	1.10	1.00	4.14	60.0	M.
		50	7.1	1.730	.....	1.840	1.420	1.630	.94	1.22	1.06	4.34	67.0	M.
		60	7.1	2.210	.....	2.460	1.800	1.760	.90	1.23	1.26	1.20 5.25	17.0 74.0	M.
		70	7.4	2.910	.....	3.220	2.200	2.640	.90	1.32	1.10	.30 5.18	4.0 70.0	M.
		80	7.8	3.390	3.430	3.450	3.070	3.600	.98	1.10	.94	4.88	62.5	M.
		100	8.5	3.200	3.220	3.240	2.830	3.460	.99	1.13	.92	5.27	62.0	M.
		110	7.9	3.010	.....	3.080	2.640	2.980	.98	1.14	1.01	5.06	64.0	M.
		130	6.4	3.140	.....	3.120	2.810	3.490	1.00	1.12	.90	3.71	58.0	M.
		150	6.3	2.830	.....	2.880	2.480	2.780	.98	1.14	1.02	3.97	63.0	M.
		170	6.4	1.430	.....	1.500	1.180	1.420	.95	1.21	1.01	4.16	65.0	M.
Mean	.....	.....	.....	.....	.....	.....	.....	.....	.97	1.09	.93	.....	64.2	

LITTLE TENNESSEE RIVER,[a] JUDSON, N. C.

Apr. 27	4.25	580	4.2	5.380	5.640	5.700	4.950	6.770	0.94	1.09	0.78	2.81	67.0	B.
		595	3.6	6.120	.....	5.950	6.200	7.330	1.03	.99	.83	1.98	55.0	B.
		615	4.0	7.450	7.080	7.500	7.210	8.210	.99	1.03	.91	2.48	62.0	B.
		637	5.0	7.280	7.800	7.900	6.480	8.270	.92	1.12	.88	3.35	67.0	B.
		645	6.0	6.010	4.340	4.500	4.690	8.470	1.34	1.28	.71	3.18	53.0	B.
June 28	3.56	520	2.7	3.840	.....	3.700	3.760	5.000	1.04	1.02	.77	1.55	57.5	B.
		530	3.8	5.235	.....	5.150	5.180	5.790	.....	1.01	.90	2.13	56.0	B.
		580	3.4	4.913	.....	4.800	4.810	5.630	1.02	1.02	.87	1.92	56.5	B.
		595	3.3	5.430	.....	5.260	5.460	6.280	1.03	.99	.86	1.75	53.0	B.
		605	3.9	5.520	.....	2.530	5.360	6.070	1.00	1.03	.91	2.36	60.5	B.
		645	3.7	5.040	.....	4.900	4.660	6.720	1.03	1.08	.75	2.15	58.0	B.
Mean	.....	.....	.....	.....	.....	.....	.....	.....	1.03	1.06	.83	.....	58.7	

[a] This station is on the wooden bridge of the Southern Railway, about one-third of a mile from Welch (flag station) or Judson P. O., N. C. The current is steady and swift; the bottom is of bowlders, from 3 inches to about 18 inches in diameter, and does not change. High floods overflow the thickly wooded left bank, but probably a very small part of the current passes through. The right bank is steep and high.

Vertical velocity measurements in 1903 on rivers in Virginia, etc.—Continued.

SUMMARY.

River and station.	Number of curves.	Coefficient for reducing to mean velocity the velocities observed at following points:			Per cent of depth at which thread of mean velocity is found.
		Six-tenths depth.	Top and bottom (mean of).	Top.	
Appomattox, Mattoax, Va	8	1.00	1.11	0.81	61.5
James, Cartersville, Va	6	1.01	1.08	.88	58.5
Roanoke, Roanoke, Va	8	.99	1.01	.81	61.9
Staunton, Randolph, Va	20	.96	1.12	.95	65.7
Dan, Madison, N. C	42	.99	1.01	.86	61.6
Dan, South Boston, Va	13	.98	1.09	.90	63.6
Reddie, North Wilkesboro, N. C	3	1.02	.97	.83	57.7
Yadkin, North Wilkesboro, N. C	33	.96	1.12	.91	66.5
Yadkin, Salisbury, N. C	10	1.02	1.06	.81	58.2
Catawba, Morganton, N. C	39	1.00	1.04	.82	60.7
Catawba, Mount Holly, N. C	2	1.00	1.08	.88	58.5
Catawba, Belmont, N. C	1	1.01	1.06	.80	58.0
Waterce, Camden, S. C	4	1.01	1.13	.90	58.4
Broad, Alston, S. C	4	.96	1.22	.89	65.4
Saluda, Waterloo, S. C	3	.97	1.03	.82	62.2
Watauga, Elizabethton, Tenn	10	.98	1.07	.82	61.4
South Fork Holston, Bluff City, Tenn	13	1.02	1.05	.80	61.6
Nolichucky, Greeneville, Tenn	22	1.02	1.02	.80	59.1
French Broad, Oldtown, Tenn	9	.99	1.07	.86	61.6
Pigeon, Newport, Tenn	4	.99	1.15	.91	61.2
Tuckasegee, Bryson City, N. C	36	.97	1.09	.93	64.2
Little Tennessee, Judson, N. C	11	1.03	1.06	.83	58.7
For all the above curves	301	1.00	1.07	.86	61.3

MEASUREMENTS ON STREAMS IN THE CATSKILL MOUNTAINS.

The results in the following tables were deduced from 378 measurements, taken in the Catskill Mountains in 1902–3,[a] at the time of regular gagings of the rivers, and with the same degree of accuracy with which the current-meter measurements were made. A small Price current meter was used, being held so that velocities were obtained at various points in regular order vertically, usually at intervals of from one-half to 1 foot. The meter in most cases hung freely in the water, but where the current was swift and tended to swing it out of the vertical, it was lowered from the downstream side of the bridge and stayed by means of a wire from the upstream side, being thus kept in the vertical plane.

The meter was held fifty seconds at each point to determine the velocity, and in each case the first reading was checked by a second. The results were plotted upon cross-section paper, the depths as ordinates and the velocities as abscissas, and a smooth curve drawn through the points. The mean velocity was then obtained by taking 10 velocities from the curve at distances of one-tenth the total depth apart and dividing their sum by ten. The velocities were taken at points 0.05, 0.15, 0.25, etc.—0.95 of the total depth on the curve.

A study of the tables for each river will explain itself. The first column gives the date on which the observations were made; the second.

[a] Results previous to this time are found in Water Sup. and Irr. Paper No. 76, U. S. Geol. Survey, 1903

the gage height at time of gaging; the third, the distance from the initial point (this being permanently marked on the bridge); the fourth, the depth of water at the measuring point; the fifth, the mean velocity in feet per second as obtained from the vertical velocity curves as previously shown; the sixth, the velocities at $\frac{6}{10}$ depth as scaled from the velocity curves; the seventh, the velocities at about 0.5 foot below the surface; the eighth and the ninth, the coefficients to be applied to the velocity columns marked "Six-tenths," and "Top," respectively, to reduce them to mean velocity; the tenth, the depth of thread of mean velocity in per cent of total depth as measured from the surface of the water; the last, the character of the bed of the stream at the point of observation.

The summary explains itself. It shows that the mean velocity of 378 determinations is found practically at a depth of 0.60 of the total depth, and that the velocity at this depth is well within a permissible error, whether obtained from the curve or from actual observation.

It appears from a study of these vertical velocity curves (1) that a rough bed causes a drag at the lower end of the vertical velocity curve and tends to raise the point of mean velocity in the section, so that a measurement at 0.6 depth would be too small; (2) that, vice versa, a smooth bed tends to make the measurement at 0.60 depth too large; (3) that the depth of a river has a marked effect on the point of mean velocity, this point in a shallow river approaching 0.55 depth, and in a deeper one approaching 0.65 depth; (4) that in general the error in using the 0.60-depth method is well within the limit of accuracy in stream-gaging work.

In the following tables G indicates gravel; S, sand or silt; R, smooth rock; B, bowlders; *that the actual $\frac{6}{10}$-measured velocity coincides with that taken from the curve.

Vertical velocity measurements on rivers in the Catskill Mountains.

CATSKILL CREEK, SOUTH CAIRO, N. Y.

Date.	Gage height.	Distance from initial point.	Depth at measuring point.	Velocity in feet per second by following methods:			Coefficients for reducing to mean velocity:		Depth of thread of mean velocity (in percent of depth).	Character of bed of stream.
				Vertical velocity.	Six-tenths.	Top.	Six-tenths.	Top.		
	Feet.	*Feet.*	*Feet.*						*Feet.*	
September 5, 1902	2.79	50.0	2.1	0.27	0.26	0.32	1.04	0.84	55.0	G.
August 27, 1902	2.74	31.5	2.2	*.56	*.56	.62	1.00	.90	60.0	R.
September 5, 1902	2.79	31.5	2.3	.63	.62	.72	1.02	.87	55.0	R.
August 27, 1902	2.74	29.0	2.5	.45	*.40	.62	1.12	.73	48.0	B.
September 5, 1902	2.79	29.0	2.5	.53	.52	.69	1.02	.77	60.0	B.
Do	2.79	25.0	2.6	.34	.31	.43	1.09	.79	55.0	B.
August 27, 1902	2.74	25.0	2.7	.30	*.30	.37	1.00	.81	60.0	B.
September 22, 1902	3.32	50.0	2.7	.72	.65	.94	1.11	.77	50.0	G.
August 13, 1902	3.49	50.0	2.9	.80	*.75	1.02	1.07	.78	52.0	G.
Do	3.49	80.0	2.9	.99	1.00	1.17	.99	.85	60.0	Small B.
August 12, 1903	2.99	50.0	3.0	.77	.75	.92	1.03	.84	57.0	R.
August 27, 1902	2.74	45.0	3.2	.40	.40	.41	1.00	.97	60.0	R.
September 22, 1902	3.32	25.0	3.2	.83	.83	.88	1.00	.94	60.0	Small B.

Vertical velocity measurements on rivers in the Catskill Mountains—Continued.

CATSKILL CREEK, SOUTH CAIRO, N. Y.—Continued.

Date.	Gage height.	Distance from initial point.	Depth at measuring point.	Velocity in feet per second by following methods:			Coefficients for reducing to mean velocity:		Depth of thread of mean velocity (in percent of depth).	Character of bed of stream
				Vertical velocity.	Six-tenths.	Top.	Six-tenths.	Top.		
	Feet.	*Feet.*	*Feet.*						*Feet.*	
August 13, 1902	3.49	25.0	3.3	0.83	0.84	1.03	0.99	0.81	60.0	Small B
September 5, 1902	2.79	45.0	3.3	.51	.50	.54	1.02	.94	60.0	R.
October 10, 1902	3.96	30.0	3.4	1.08	1.10	1.25	.98	.86	61.0	R
August 27, 1902	2.74	40.0	3.5	.65	.65	.72	1.00	.90	60.0	R
September 5, 1902	2.79	40.0	3.6	.72	.70	.81	1.03	.89	56.0	R.
September 12, 1903	2.99	45.0	3.6	.81	.80	.97	1.01	.83	59.0	R.
Do	2.99	40.0	3.7	.78	.77	.85	1.01	.92	58.0	G.
October 10, 1902	3.96	60.0	3.8	1.08	1.08	1.42	1.00	.76	60.0	G.
August 27, 1902	2.74	35.0	3.9	.43	*.43	.63	1.00	.68	60.0	R.
October 10, 1902	3.96	55.0	3.9	1.12	1.08	1.46	1.03	.77	57.0	G.
April 11, 1902	6.86	90.0	4.0	3.91	3.90	4.47	1.00	.87	60.0	G.
May 10, 1902	3.47	45.0	4.0	.95	.93	1.22	1.02	.78	58.0	R.
August 13, 1902	3.49	45.0	4.0	1.00	.93	1.26	1.08	.79	50.0	R.
September 22, 1902	3.32	45.0	4.0	.93	.85	1.15	1.10	.81	50.0	R.
October 10, 1902	3.96	25.0	4.0	.83	*.80	1.08	1.03	.77	55.0	R.
Do	3.96	50.0	4.1	1.18	1.17	1.42	1.01	.83	59.0	G.
September 9, 1902	3.81	40.0	4.1	1.00	.98	1.16	1.02	.86	59.0	G.
July 9, 1902	3.36	40.0	4.2	1.09	1.09	1.23	1.00	.89	60.0	R.
August 13, 1902	3.49	40.0	4.2	1.17	1.14	1.32	1.02	.89	52.0	R
September 5, 1902	2.79	35.0	4.2	.53	.53	.72	1.00	.74	60.0	R
September 22, 1902	3.32	40.0	4.2	1.14	1.12	1.29	1.02	.88	55.0	R
August 12, 1903	2.99	35.0	4.2	.45	.42	.66	1.07	.68	55.0	G
April 11, 1902	6.86	70.0	4.6	3.76	3.87	4.42	.97	.85	63.0	G.
May 10, 1902	3.47	35.0	4.6	.98	.90	1.27	1.03	.73	59.0	R
October 10, 1902	3.96	45.0	4.6	1.19	1.17	1.41	1.02	.84	57.0	R.
September 22, 1902	3.32	35.0	4.7	.92	.90	1.15	1.02	.80	59.0	R.
April 22, 1902	4.06	40.0	4.8	1.79	1.74	2.07	1.03	.86	58.0	R.
August 13, 1902	3.49	35.0	4.9	.99	1.00	1.27	.99	.78	60.0	R
October 10, 1902	3.96	40.0	4.9	1.14	1.14	1.34	1.00	.85	60.0	R
August 2, 1902	5.36	30.0	5.2	2.49	2.45	3.00	1.02	.83	57.0	R.
October 10, 1902	3.96	35.0	5.4	.98	.98	1.21	1.00	.81	60.0	R.
August 2, 1902	5.36	40.0	6.1	3.09	*3.20	3.24	.96	.95	67.0	R.
April 11, 1902	6.86	30.0	6.6	3.08	2.98	3.64	1.03	.85	55.0	G.
Do	6.86	50.0	7.0	3.56	3.65	4.12	.97	.86	64.0	R.
Mean							1.02	.83	57.7	

DELAWARE RIVER (EAST BRANCH), HANCOCK, N. Y.

May 29, 1903	2.72	320	2.1	0.92	0.91	1.20	1.01	0.77	60	B.
Do	2.72	230	2.4	.75	.70	1.02	1.07	.74	57	G.
Do	2.72	240	2.5	1.05	1.04	1.22	1.01	.86	60	B
September 1, 1903	4.96	90	3.0	3.23	3.14	4.04	1.03	.80	58	G
Do	4.96	320	4.4	3.65	3.58	4.51	1.02	.81	59	B
Do	4.96	240	4.8	4.12	4.13	4.46	1.00	.92	60	B.
Mean							1.023	.816	59	

DELAWARE RIVER (WEST BRANCH), HANCOCK, N. Y.

July 21, 1903	3.10	140	1.7	1.67	1.60	2.05	1.04	0.82	59	G
Do	3.10	160	1.9	2.06	2.00	2.45	1.03	.84	58	G
Do	3.10	170	2.0	1.93	2.00	2.25	.96	.86	62	G
Do	3.10	180	2.2	2.40	2.30	3.07	1.04	.78	59	G
Do	3.10	190	2.5	2.78	2.74	2.90	1.01	.95	58	G
August 31, 1903	5.35	220	3.1	3.10	3.00	3.90	1.03	.80	55	G
Do	5.35	60	3.2	3.07	3.05	3.78	1.01	.81	60	G
Do	5.35	80	3.9	3.80	3.65	4.74	1.04	.80	56	G
Do	5.35	130	3.9	4.26	4.25	5.17	1.00	.82	60	G
Do	5.35	210	3.9	3.82	3.68	4.70	1.04	.81	55	G
Do	5.35	150	4.1	4.67	4.60	5.40	1.01	.86	55	G
Do	5.35	200	4.2	4.45	4.40	5.32	1.01	.83	59	G
Mean							1.018	.831	58	

Vertical velocity measurements on rivers in the Catskill Mountains—Continued.

ESOPUS CREEK, KINGSTON, N. Y.

Date.	Gage height.	Distance from initial point.	Depth at measuring point.	Velocity in feet per second by following methods:			Coefficients for reducing to mean velocity:		Depth of thread of mean velocity (in percent of depth).	Character of bed of stream.
				Vertical velocity.	Six-tenths.	Top.	Six-tenths.	Top.		
	Feet.	*Feet.*	*Feet.*						*Feet.*	
August 21, 1902	4.94	40	1.9	0.74	*0.72	0.82	1.03	0.90	54.0	B.
July 22, 1903	5.17	35	2.1	.96	.95	1.18	1.00	.82	60.0	B.
May 24, 1902	5.02	40	2.2	.79	*.76	1.03	1.04	.77	58.0	B.
Do	5.02	70	2.4	1.06	1.06	1.32	1.00	.80	60.0	B.
July 22, 1903	5.17	40	2.6	1.04	1.01	1.16	1.03	.90	57.0	B.
Do	5.17	70	2.6	1.33	1.33	1.51	1.00	.88	60.0	B.
September 23, 1902	5.87	60	2.9	1.59	1.52	2.03	1.05	.78	55.0	B.
July 9, 1902	5.81	40	3.0	1.29	1.29	1.50	1.00	.86	60.0	B.
August 24, 1903	5.72	40	3.0	1.37	1.34	1.58	1.02	.87	59.0	B.
September 23, 1902	5.87	40	3.1	1.19	1.19	1.45	1.00	.82	60.0	B.
			3.2							
			3.4							
August 21, 1902	4.94	75	3.5	.99	*1.02	1.18	.97	.84	60.0	B.
June 6, 1902	4.48	75	3.6	.65	.64	.86	1.00	.76	60.0	B.
August 12, 1902	6.28	40	3.6	1.48	*1.42	1.77	1.04	.84	55.0	B.
September 4, 1902	4.49	75	3.7	.66	*.60	.91	1.10	.73	53.0	B.
July 16, 1902	5.13	75	4.1	1.12	1.11	1.41	1.00	.80	60.0	B.
July 22, 1903	5.17	75	4.1	1.16	1.15	1.42	1.00	.82	60.0	B.
May 24, 1902	5.02	100	4.4	.73	*.78	.86	.93	.85	66.0	S.
July 22, 1903	5.17	100	4.6	1.21	1.20	1.34	1.00	.90	60.0	S.
July 30, 1902	7.65	60	4.8	2.37	*2.36	2.48	1.00	.80	60.0	B.
September 4, 1903	7.66	40	4.9	2.63	2.74	2.99	.96	.88	65.0	B.
May 14, 1902	5.83	100	5.0	1.19	1.24	1.29	.96	.92	67.0	S.
August 12, 1902	6.28	75	5.0	1.67	1.64	2.15	1.02	.78	59.0	B.
June 26, 1902	4.98	80	5.2	1.09	1.08	1.22	1.00	.90	60.0	B.
July 24, 1902	8.11	60	5.2	2.71	*2.71	3.07	1.00	.89	60.0	B.
August 21, 1902	4.94	85	5.2	.96	.97	1.03	.99	.93	61.0	S.
May 24, 1902	5.02	80	5.4	1.01	*1.00	1.16	1.00	.87	60.0	B.
September 4, 1902	4.49	85	5.4	.74	*.74	.84	1.00	.88	60.0	S.
July 22, 1903	5.17	80	5.4	1.24	1.23	1.44	1.00	.86	60.0	B.
August 24, 1903	5.72	80	5.4	1.47	1.47	1.72	1.00	.85	60.0	B.
July 22, 1903	5.17	85	5.5	1.17	1.18	1.30	.99	.90	61.0	B.
September 23, 1902	5.87	80	5.6	1.56	*1.55	1.74	1.00	.90	60.0	B.
July 22, 1903	5.17	95	5.7	1.16	1.13	1.37	1.03	.85	58.0	S.
June 16, 1902	4.48	90	5.8	.70	.69	.80	1.00	.87	60.0	S.
May 14, 1902	5.83	80	6.0	1.38	*1.40	1.59	.99	.87	62.0	S.
October 4, 1902	9.32	60	6.1	2.83	2.85	3.02	.99	.94	61.0	B.
August 21, 1902	4.94	90	6.2	.89	.87	1.06	1.02	.84	59.0	S.
July 22, 1903	5.17	90	6.2	1.12	1.05	1.44	1.07	.78	55.0	S.
July 16, 1902	5.13	90	6.3	1.07	*1.02	1.23	1.05	.87	58.0	S.
August 12, 1902	6.28	85	6.3	1.71	*1.71	1.71	1.00	1.00	60.0	S.
October 4, 1902	9.32	40	6.3	3.12	3.23	3.16	.97	.99	66.0	B.
May 24, 1902	5.02	90	6.4	.90	*.89	1.10	1.00	.82	60.0	S.
April 10, 1902	13.37	15	6.8	5.04	*5.12	5.16	.98	.97	70.0	B.
May 14, 1902	5.83	90	7.2	1.22	*1.22	1.38	1.00	.88	60.0	S.
July 9, 1902	5.81	90	7.2	1.30	*1.28	1.50	1.00	.86	60.0	S.
September 4, 1903	7.66	90	7.6	2.28	2.23	2.67	1.02	.83	59.0	S.
April 10, 1902	13.37	20	7.8	4.89	5.07	5.12	.97	.95	74.0	B.
			8.7							
			8.8							
October 4, 1902	9.32	80	9.0	2.75	2.77	2.80	.98	.98	65.0	B.
April 10, 1902	13.37	30	9.6	4.94	5.11	5.18	.97	.95	75.0	B.
			9.6							
April 10, 1902	13.37	50	10.2	4.56	4.75	4.83	.96	.94	70.0	B.
Do	13.37	60	10.5	4.48	4.71	4.66	.95	.96	75.0	B.
Do	13.37	40	11.0	4.70	4.90	4.97	.96	.94	70.0	B.
Do	13.37	70	11.0	4.34	4.55	4.40	.96	.98	75.0	B.
Do	13.37	100	13.0							
Do	13.37	80	14.0	4.09	4.25	4.12	.96	.99	69.0	B., S.
Mean							.999	.875	61.7	

Vertical velocity measurements on rivers in the Catskill Mountains—Continued.

ESOPUS CREEK, OLIVEBRIDGE, N. Y.

Date.	Gage height.	Distance from initial point.	Depth at measuring point.	Velocity in feet per second by following methods:			Coefficients for reducing to mean velocity:		Depth of thread of mean velocity (in percent of depth).	Character of bed of stream.
				Vertical velocity.	Six-tenths.	Top.	Six-tenths.	Top.		
	Feet.	*Feet.*	*Feet.*						*Feet.*	
August 11, 1903	1.70	25	0.9	1.59	1.60	1.54	0.99	1.04	61.0	R.
Do	1.70	30	1.0	1.74	1.77	1.84	.98	.95	62.0	R.
July 9, 1903	1.60	160	1.2	.50	.48	.61	1.04	.82	57.0	R.
Do	1.60	90	1.3	1.01	1.00	1.09	1.01	.92	60.0	R.
August 11, 1903	1.70	160	1.3	.51	.51	.54	1.00	.94	60.0	R.
September 8, 1903	1.81	70	1.3	2.59	2.53	2.86	1.02	.91	54.0	R.
Do	1.81	80	1.3	3.18	3.15	3.33	1.01	.96	55.0	R.
June 2, 1903	1.10	40	1.4	1.36	1.34	1.58	1.01	.86	57.0	G.
July 9, 1903	1.60	130	1.4	.72	.72	.78	1.00	.81	60.0	R.
July 2, 1903	2.14	160	1.5	.98	.94	1.13	1.04	.87	55.0	R.
Do	2.14	70	1.6	2.70	2.74	2.86	.99	.94	60.0	R.
Do	2.14	165	1.6	.99	.96	1.11	1.03	.90	55.0	R.
July 4, 1903	1.92	130	1.6	1.49	1.42	1.68	1.05	.90	57.0	R.
July 9, 1903	1.60	35	1.6	1.62	1.60	1.75	1.01	.93	59.0	G.
August 11, 1903	1.70	130	1.6	1.17	1.15	1.22	1.02	.95	59.0	R.
August 14, 1903	1.62	35	1.6	1.41	1.35	1.66	1.04	.85	55.0	G.
July 2, 1903	2.14	80	1.7	3.02	3.04	3.09	.99	.97	61.0	R.
July 9, 1903	1.60	50	1.7	1.69	1.70	1.79	.99	.95	60.0	R.
Do	1.60	60	1.7	1.34	1.25	1.62	1.07	.82	56.0	R.
Do	1.60	120	1.8	.53	.52	.75	1.02	.70	59.0	R.
Do	1.60	150	1.8	.42	.41	.54	1.03	.78	60.0	R.
August 11, 1903	1.70	35	1.8	1.65	1.65	1.89	1.00	.87	60.0	G.
Do	1.70	60	1.8	1.76	1.74	1.91	1.01	.93	57.0	R.
Do	1.70	90	1.8	1.63	1.60	1.81	1.02	.90	59.0	R.
September 8, 1903	1.81	55	1.8	1.86	1.83	2.13	1.02	.87	58.0	R.
July 2, 1903	2.14	130	1.9	1.98	1.96	2.13	1.01	.93	55.0	R.
August 11, 1903	1.70	50	1.9	1.73	1.75	1.93	.99	.90	60.0	R.
September 8, 1903	1.81	35	1.9	1.61	1.65	2.08	.98	.78	61.0	G.
July 9, 1903	1.60	40	2.0	1.46	1.45	1.56	1.01	.94	60.0	G.
Do	1.60	140	2.0	.48	.50	.51	.96	.94	61.0	R.
August 11, 1903	1.70	150	2.0	.68	.69	.71	.98	.96	61.0	R.
August 14, 1903	1.62	40	2.0	1.47	1.45	1.63	1.01	.90	59.0	G.
September 2, 1903	2.74	30	2.0	3.58	3.53	3.61	1.00	.97	60.0	R.
July 2, 1903	2.14	35	2.1	2.32	2.25	2.67	1.03	.87	55.0	G.
July 9, 1903	1.60	45	2.1	1.62	1.57	1.86	1.03	.87	57.0	R.
August 11, 1903	1.70	140	2.1	.90	.86	1.11	1.05	.82	53.0	R.
August 14, 1903	1.62	140	2.1	.65	.63	.80	1.03	.81	57.0	R.
September 2, 1903	2.74	80	2.1	4.47	4.44	4.79	1.01	.94	57.0	R.
September 8, 1903	1.81	50	2.1	1.91	1.92	2.17	1.00	.90	60.0	R.
August 11, 1903	1.70	40	2.2	1.61	1.57	1.79	1.02	.90	55.0	G.
August 14, 1903	1.62	45	2.2	1.60	1.55	1.84	1.03	.87	55.0	R.
September 8, 1903	1.81	40	2.2	1.27	1.30	1.63	.98	.80	62.0	G.
July 2, 1903	2.14	60	2.3	2.73	2.62	3.09	1.04	.88	56.0	R.
Do	2.14	120	2.3	1.30	1.27	1.61	1.02	.80	56.0	R.
Do	2.14	150	2.3	1.35	1.33	1.49	1.01	.91	56.0	R.
July 4, 1903	1.92	140	2.3	1.15	1.10	1.32	1.04	.88	52.0	R.
September 8, 1903	1.81	140	2.3	1.13	1.10	1.32	1.03	.85	54.0	R.
July 2, 1903	2.14	40	2.4	2.09	2.05	2.52	1.02	.83	57.0	G.
July 4, 1903	1.92	50	2.4	2.05	2.00	2.27	1.02	.90	57.0	R.
August 11, 1903	1.70	45	2.4	1.63	1.62	1.72	1.00	.94	60.0	R.
September 8, 1903	1.81	45	2.4	1.85	1.85	2.05	1.00	.90	60.0	R.
July 2, 1903	2.14	140	2.5	1.58	1.53	1.86	1.03	.86	52.0	R.
Do	2.14	50	2.6	2.50	2.46	2.74	1.02	.91	58.0	R.
September 2, 1903	2.74	90	2.6	4.09	4.10	4.41	1.00	.93	60.0	R.
Do	2.74	130	2.6	2.77	2.76	3.04	1.00	.91	59.0	R.
July 2, 1903	2.14	100	2.9	2.00	1.98	2.08	1.01	.96	55.0	R.
September 2, 1903	2.74	150	2.9	3.02	3.03	3.31	1.00	.91	60.0	R.
June 17, 1903	2.67	50	3.0	4.16	4.13	4.46	1.01	.93	55.0	R.
September 2, 1903	2.74	35	3.0	3.91	3.89	4.39	1.01	.89	59.0	G.
Do	2.74	50	3.0	4.25	4.22	4.41	1.01	.97	55.0	R.
Do	2.74	140	3.1	2.71	2.59	3.61	1.05	.75	54.0	R.
Do	2.74	40	3.2	3.74	3.69	4.27	1.02	.88	57.0	G.
June 13, 1903	3.08	150	3.3	4.06	4.02	4.37	1.01	.93	56.0	R.
October 11, 1903	4.05	30	3.4	4.39	4.30	4.92	1.02	.89	54.0	R.
Do	4.05	100	4.8	4.68	4.65	5.10	1.01	.91	57.0	R.
October 10, 1903	5.80	30	5.1	5.86	5.95	5.96	.98	.98	65.0	R.
Do	5.05	35	5.1	6.16	6.12	7.05	1.01	.87	59.0	G.
Do	5.80	165	5.2	7.62	7.67	7.97	.99	.95	62.0	R.
Mean	.....	.....	.....	.....	.....	.....	1.012	.894	57.7	

Vertical velocity measurements on rivers in the Catskill Mountains—Continued.

FISHKILL CREEK, GLENHAM, N. Y.

Date.	Gage height.	Distance from initial point.	Depth at measuring point.	Velocity in feet per second by following methods:			Coefficients for reducing to mean velocity:		Depth of thread of mean velocity (in percent of depth).	Character of bed of stream.
				Vertical velocity.	Six-tenths.	Top.	Six-tenths.	Top.		
	Feet.	*Feet.*	*Feet.*						*Feet.*	
August 26, 1902	3.04	80	2.1	0.26	0.26	0.30	1.00	0.87	60.0	G.
Do	3.04	65	2.2	.37	.33	.52	1.12	.71	55.0	G.
Do	3.04	75	2.2	.27	.25	.39	1.08	.69	54.0	G.
July 28, 1902	3.90	90	2.3	.96	1.03	1.08	.89	.89	66.0	G.
August 26, 1902	3.04	70	2.5	.37	.35	.49	1.06	.76	55.0	G.
Do	3.04	40	2.6	.29	.29	.32	1.00	.91	60.0	G.
August 11, 1902	3.51	80	2.7	.61	.55	.85	1.10	.72	50.0	G.
Do	3.51	65	2.8	.74	.76	.97	.97	.76	62.0	G.
August 26, 1902	3.04	60	2.9	.46	.48	.56	.96	.82	65.0	G.
Do	3.04	55	3.0	.45	.44	.56	1.00	.80	56.0	G.
July 28, 1902	3.90	65	3.1	.99	.99	1.37	1.00	.72	60.0	G.
August 26, 1902	3.04	45	3.3	.37	.37	.39	1.00	.95	60.0	G.
July 28, 1903	3.55	40	3.3	.64	.63	.80	1.00	.80	60.0	G.
Do	3.55	60	3.4	.77	.77	.90	1.00	.86	60.0	G.
August 26, 1902	3.04	50	3.5	.38	.38	.49	1.00	.78	60.0	G.
July 28, 1903	3.55	50	3.7	.73	.70	.97	1.04	.75	56.0	G.
August 11, 1902	3.51	45	3.8	.73	.70	.92	1.04	.79	55.0	G.
September 5, 1903	4.52	70	3.9	1.60	1.58	1.96	1.01	.82	60.0	G.
August 11, 1902	3.51	50	4.0	.77	.79	.94	.98	.82	62.0	G.
July 28, 1902	3.90	45	4.1	1.07	1.04	1.29	1.03	.83	56.0	G.
September 5, 1903	4.52	60	4.4	1.65	1.67	1.81	.99	.91	62.0	G.
Do	4.52	50	5.0	1.75	1.70	2.05	1.03	.85	57.0	G.
Mean							1.036	.79	58.7	

HOUSATONIC RIVER, GAYLORDSVILLE, CONN.

August 5, 1902	4.28	130	2.1	1.79	1.79	2.27	1.00	0.79	60.0	G., B.
July 24, 1903	5.20	180	2.1	.67	.62	.99	1.08	.68	55.0	B.
Do	5.20	160	2.1	3.19	3.10	3.73	1.03	.85	58.0	B.
Do	5.20	150	2.2	3.42	3.30	4.08	1.03	.84	54.0	B.
August 5, 1902	4.28	120	2.5	1.30	1.27	1.64	1.02	.79	58.0	G., B.
July 24, 1903	5.20	170	2.6	2.37	2.35	2.76	1.00	.86	60.0	B.
August 20, 1902	3.95	100	2.7	2.04	1.97	2.42	1.03	.84	54.0	G., B.
September 8, 1902	3.45	90	2.7	1.67	1.58	2.10	1.05	.80	56.0	G., B.
June 23, 1902	4.46	110	2.8	2.22	2.19	2.54	1.01	.87	57.0	G., B.
September 8, 1902	3.45	40	2.8	1.65	1.65	1.95	1.00	.85	60.0	G.
July 24, 1903	5.20	140	2.8	3.41	3.38	3.90	1.01	.87	59.0	B.
August 5, 1902	4.28	90	3.1	2.21	2.25	2.83	.98	.78	61.0	G., B.
July 24, 1903	5.20	130	3.1	2.72	2.54	3.82	1.07	.71	58.0	G., B.
June 23, 1902	4.46	30	3.2	1.96	1.96	2.28	.99	.86	60.0	G.
September 19, 1902	3.75	40	3.2	1.89	1.87	2.15	1.01	.88	59.0	G.
August 20, 1902	3.95	40	3.4	2.08	2.07	2.36	1.00	.88	59.0	G.
July 24, 1903	5.20	120	3.6	2.74	2.76	3.61	.99	.76	60.0	G., B.
June 23, 1902	4.46	90	3.7	2.59	2.59	3.07	1.00	.84	60.0	G., B.
August 5, 1902	4.28	40	3.7	2.11	2.03	2.54	1.04	.83	55.0	G.
July 24, 1903	5.20	20	3.8	1.79	1.77	2.11	1.00	.85	60.0	G.
Do	5.20	110	3.8	3.03	2.86	3.85	1.06	.79	56.0	G., B.
June 23, 1902	4.46	50	4.0	2.89	2.91	3.18	.99	.91	61.0	G.
September 8, 1902	3.45	70	4.0	2.07	2.00	2.47	1.03	.84	55.0	G.
July 24, 1903	5.20	100	4.3	3.36	3.22	4.27	1.04	.79	55.0	G., B.
Do	5.20	30	4.4	2.29	2.37	2.36	.96	.97	63.0	G.
September 15, 1903	3.90	70	4.6	2.37	2.35	2.93	1.00	.81	60.0	G.
Do	3.90	60	4.7	2.27	2.26	2.84	1.00	.80	60.0	G.
July 24, 1903	5.20	90	4.8	3.05	2.88	4.20	1.06	.73	55.0	G., B.
August 5, 1902	4.28	70	4.9	2.59	2.54	3.05	1.02	.85	57.0	G.
Do	4.28	60	4.9	2.59	2.50	3.17	1.04	.82	52.0	G.
June 23, 1902	4.46	70	5.0	2.83	2.86	3.26	.99	.87	63.0	G.
July 24, 1903	5.20	40	5.0	3.00	3.17	3.23	.95	.93	66.0	G.
Do	5.20	50	5.4	3.45	3.48	4.08	.99	.85	61.0	G.
Do	5.20	80	5.5	3.48	3.40	4.32	1.02	.81	59.0	G.
Do	5.20	60	6.2	3.66	3.70	4.27	1.00	.86	60.0	G.
Do	5.20	70	6.2	3.60	3.58	4.41	1.00	.82	60.0	G.
Mean							1.01	.83	58.5	

Vertical velocity measurements on rivers in the Catskill Mountains—Continued.

RONDOUT CREEK, ROSENDALE, N. Y.

Date.	Gage height.	Distance from initial point.	Depth at measuring point.	Velocity in feet per second by following methods:			Coefficients for reducing to mean velocity:		Depth of thread of mean velocity (in percent of depth).	Character of bed of stream.
				Vertical velocity.	Six-tenths.	Top.	Six-tenths.	Top.		
	Feet.	*Feet.*	*Feet.*						*Feet.*	
August 7, 1902	7.39	35	4.4	1.26	1.30	1.45	0.97	0.87	65.0	B.
July 29, 1902	7.40	35	4.5	1.34	1.40	1.49	.96	.90	65.0	B.
September 5, 1903	7.24	40	5.1	1.68	1.67	1.77	1.00	.95	60.0	B.
August 21, 1902	6.33	75	5.3	.39	.40	.49	1.00	.80	61.0	B.
July 18, 1902	6.33	75	5.5	.36	.35	.44	1.00	.82	58.0	B.
August 29, 1902	6.33	100	5.5	.35	.37	.39	.95	.90	67.0	B.
Do	6.33	75	5.6	.42	.44	.51	.96	.82	62.0	B.
August 21, 1902	6.33	85	5.9	.38	.38	.50	1.00	.76	60.0	B.
August 29, 1902	6.33	85	5.9	.44	.44	.51	1.00	.86	60.0	B.
July 18, 1902	6.33	90	6.0	.35	.33	.44	1.06	.79	53.0	B.
August 21, 1902	6.33	90	6.1	.39	.37	.49	1.06	.80	55.0	B.
August 29, 1902	6.33	90	6.1	.41	.41	.48	1.00	.85	60.0	B.
September 5, 1903	7.24	100	6.3	1.17	1.22	1.49	.96	.79	63.0	B.
March 21, 1902	8.07	40	6.4	2.42	2.46	2.73	.98	.89	60.0	B.
August 7, 1902	7.39	75	6.7	1.64	1.66	2.15	.99	.77	61.0	B.
July 29, 1902	7.40	75	6.8	1.73	1.75	2.18	.99	.79	61.0	B.
September 5, 1903	7.24	70	6.8	1.64	1.66	2.15	.99	.76	61.0	B.
Do	7.24	60	7.2	1.73	1.82	1.96	.95	.88	68.0	B.
Do	7.24	80	7.3	1.64	1.70	1.91	.97	.86	65.0	B.
Do	7.24	90	7.3	1.36	1.41	1.63	.97	.83	63.0	B.
March 21, 1902	8.07	60	8.2	2.55	2.65	2.80	.97	.91	61.0	B.
Mean							.987	.84	61.4	

SCHOHARIE CREEK, PRATTSVILLE, N. Y.

June 4, 1903	4.72	12	1.1	1.42	1.42	1.77	1.00	0.80	60.0	S.
May 15, 1903	5.02	40	4.2	.52	.53	.68	.98	.77	60.0	S.
Do	5.02	30	4.7	.64	.63	.83	1.02	.77	60.0	S.
August 26, 1903	6.29	60	4.8	1.59	1.60	1.78	.99	.89	60.0	S.
June 30, 1903	5.98	50	5.0	1.66	1.66	2.03	1.00	.82	60.0	S.
August 26, 1903	6.29	40	5.2	2.07	2.06	2.55	1.01	.81	60.0	S.
May 15, 1903	5.02	20	5.6	.56	.53	.76	1.05	.74	50.0	S.
June 30, 1903	5.98	30	5.7	1.98	1.96	2.43	1.01	.81	59.0	S.
August 26, 1903	6.29	30	5.8	2.28	2.22	2.76	1.02	.83	58.0	S.
June 30, 1903	5.98	20	6.3	1.78	1.72	2.31	1.03	.77	57.0	S.
August 30, 1903	7.62	40	6.6	3.20	3.05	4.22	1.05	.76	57.0	S.
Mean							1.014	.797+	58.3	

TENMILE RIVER, DOVER PLAINS, N. Y.

August 4, 1902	4.69	11	1.8	0.46	0.46	0.67	1.00	0.69	60.0	B.
Do	4.69	35	2.2	1.28	1.23	1.43	1.02	.88	58.0	R.
Do	4.69	40	2.4	1.21	1.20	1.45	1.00	.83	60.0	R.
Do	4.69	45	2.8	1.24	1.24	1.36	1.00	.91	60.0	R., S.
July 23, 1903	6.18	10	3.0	1.10	1.12	1.38	.98	.80	62.0	S.
September 14, 1903	4.88	55	3.0	1.10	1.08	1.30	1.02	.85	55.0	
August 4, 1902	4.69	15	3.1	.66	.72	.84	.92	.79	66.0	B.
August 19, 1902	4.28	50	3.4	.64	.64	.66	1.00	.97	60.0	S.
July 23, 1903	6.18	35	3.4	1.98	1.98	2.13	1.00	.93	60.0	S.
Do	6.18	40	3.5	1.97	2.05	2.12	.96	.93	62.0	S.
October 2, 1902	5.80	40	3.6	1.74	1.69	2.03	1.03	.86	55.0	R.
September 2, 1902	3.95	55	3.7	.24	.25	.24	.96	1.00	65.0	S.
August 4, 1902	4.69	22	3.8	.88	.91	.99	.97	.89	65.0	B., S.
August 19, 1902	4.48	55	3.8	.51	.51	.51	1.00	1.00	60.0	S.
August 4, 1902	4.69	50	3.9	1.14	1.13	1.27	1.00	.90	60.0	S.
July 23, 1903	6.18	50	3.9	1.94	2.02	2.13	.96	.91	67.0	S.
Do	6.18	45	4.0	1.79	1.88	2.26	.95	.79	63.0	S.
September 2, 1902	3.95	20	4.1	.60	.60	.74	1.00	.81	60.0	B.
August 4, 1902	4.69	55	4.2	.96	.96	1.04	1.00	.92	60.0	S.
August 19, 1902	4.28	20	4.2	.70	.74	.79	.95	.88	66.0	B.
September 11, 1903	4.88	25	4.2	1.11	1.06	1.42	1.06	.78	55.0	S.
August 4, 1902	4.69	30	4.3	1.07	1.05	1.33	1.02	.80	58.0	S.
July 23, 1903	6.18	55	4.3	1.94	1.93	1.98	1.00	.98	60.0	S.
Do	6.18	15	4.4	1.53	1.57	1.75	.98	.87	61.0	S.
August 4, 1902	4.69	20	4.7	.82	.86	.88	.96	.93	70.0	B.

Vertical velocity measurements on rivers in the Catskill Mountains—Continued.

TENMILE RIVER, DOVER PLAINS, N. Y.—Continued.

Date.	Gage height.	Distance from initial point.	Depth at measuring point.	Velocity in feet per second by following methods:			Coefficients for reducing to mean velocity:		Depth of thread of mean velocity (in per cent of depth).	Character of bed of stream.
				Vertical velocity.	Six-tenths.	Top.	Six-tenths.	Top.		
	Feet.	*Feet.*	*Feet.*						*Feet.*	
July 23, 1903	6.18	80	4.8	1.81	1.80	1.96	1.00	0.92	59.0	S.
August 4, 1902	4.69	60	5.0	.67	.69	.69	.97	.97	65.0	S.
October 2, 1902	5.80	50	5.3	1.75	1.73	1.93	1.03	.90	58.0	S.
July 23, 1903	6.18	20	5.3	1.72	1.72	1.98	1.00	.87	60.0	S.
Do	6.18	25	5.3	1.76	1.70	2.08	1.04	.85	56.0	S.
October 2, 1902	5.80	20	5.9	1.36	1.37	1.61	1.00	.84	60.0	B.
Do	5.80	60	6.0	1.50	1.55	1.45	.97	1.03	67.0	S.
July 21, 1902	7.18	25	7.3	2.15	2.18	2.44	.99	.88	63.0	S.
Mean							.992	.884	60.9	

WALLKILL RIVER, NEWPALTZ, N. Y.

August 28, 1902	5.86	40	3.3	0.32	0.32	0.34	1.00	0.94	60.0	S.
Do	5.86	45	3.3	.31	.32	.33	.97	.94	65.0	S.
September 18, 1902	5.96	40	3.4	.39	.42	.43	.93	.91	66.0	S.
Do	5.96	50	3.4	.41	.46	.41	.89	1.00	74.0	S.
August 15, 1902	6.72	30	3.6	.63	.62	.76	1.01	.83	59.0	S.
June 6, 1902	6.40	40	3.8	.73	.72	.84	1.01	.87	55.0	S.
May 13, 1902	6.68	45	4.0	.96	.98	1.09	.98	.88	65.0	S.
August 15, 1902	6.72	40	4.0	.96	.97	1.03	.99	.85	65.0	S.
August 6, 1902	7.98	30	5.2	1.19	1.15	1.53	1.03	.78	56.0	S.
Do	7.98	40	5.6	1.70	1.70	1.91	1.00	.89	60.0	S.
August 28, 1902	5.86	75	5.7	.37	.42	.40	.88	.92	74.0	S.
May 1, 1902	10.60	25	5.8	2.08	2.24	2.41	.93	.86	66.0	S.
September 18, 1902	5.96	75	5.8	.46	.50	.51	.92	.90	76.0	S.
August 15, 1903	7.85	40	5.8	1.70	1.78	1.70	.96	1.00	68.0	S.
Do	7.85	50	5.9	1.61	1.66	1.68	.97	.96	68.0	S.
September 18, 1902	5.96	90	6.2	.43	.43	.56	1.00	.77	60.0	S.
August 15, 1902	6.72	75	6.6	1.05	1.05	1.22	1.00	.86	60.0	S.
May 1, 1902	10.60	30	7.6	2.19	2.22	2.52	.99	.87	62.0	S.
June 19, 1902	6.18	105	7.6	.57	.62	.63	.92	.91	68.0	S.
September 18, 1902	5.96	110	7.7	.53	.60	.46	.88	1.15	79.0	S.
May 1, 1902	10.52	40	7.8	2.81	2.92	2.78	.97	1.01	72.0	S.
Do	10.52	45	8.0	2.77	2.79	2.86	.99	.97	64.0	S.
Do	10.46	50	8.0	2.74	2.76	2.88	.99	.95	64.0	S.
Do	10.46	55	8.0	2.40	2.48	2.95	.97	.81	65.0	S.
Do	10.46	60	8.2	1.97	2.02	2.98	.98	.66	62.0	S.
May 13, 1902	6.68	105	8.2	1.04	1.04	1.09	1.00	.95	60.0	S.
Do	6.68	120	8.2	.87	.42	.38	.88	.97	69.0	S.
June 6, 1902	6.40	110	8.2	.81	.80	.93	1.01	.87	60.0	S.
Do	6.40	120	8.2	.18	.19	.19	.99	.95	70.0	S.
August 15, 1903	7.85	90	8.3	1.78	1.79	2.10	1.00	.85	60.0	S.
August 15, 1902	6.72	110	8.4	1.12	1.12	1.17	1.00	0.96	60.0	S.
June 6, 1902	6.40	115	8.6	.55	.53	.69	1.01	.80	55.0	S.
May 1, 1902	10.38	65	9.4	2.00	1.83	3.02	1.09	.66	55.0	S.
June 16, 1903	15.36	20	9.7	3.07	3.05	3.11	1.01	.98	59.0	S.
May 1, 1902	10.38	70	10.4	2.59	2.59	3.05	1.00	.85	60.0	S.
Do	10.24	75	10.4	2.59	2.57	3.02	1.01	.86	59.0	S.
Do	10.24	90	10.6	2.61	2.60	2.98	1.00	.87	60.0	S.
Do	10.24	80	10.8	2.60	2.70	3.02	.96	.86	67.0	S.
Do	10.24	85	10.8	2.68	2.69	3.10	1.00	.87	60.0	S.
Do	10.24	95	11.0	2.68	2.68	3.05	1.00	.88	60.0	S.
Do	10.16	100	11.2	2.74	2.78	3.02	.98	.91	62.0	S.
Do	10.16	105	11.8	2.85	2.96	3.10	.96	.92	68.0	S.
Do	10.16	110	11.8	3.00	3.09	2.98	.97	1.01	75.0	S.
June 16, 1903	15.36	60	11.8	4.15	4.17	4.37	.99	.95	61.0	S.
May 1, 1902	10.16	115	12.4	2.84	2.92	2.90	.97	.98	69.0	S.
June 16, 1903	15.36	70	13.4	4.15	4.18	4.56	.99	.91	61.0	S.
March 24, 1903	14.95	80	15.0	4.05	4.08	4.20	.99	.96	64.0	S.
June 16, 1903	15.36	80	15.3	4.40	4.50	4.56	.98	.96	67.0	S.
Do	15.36	90	15.8	4.41	4.44	4.61	.99	.95	61.0	S.
Do	15.36	100	16.3	4.54	4.55	4.84	1.00	.94	60.0	S.
Do	15.36	120	16.4	4.05	3.97	4.65	1.02	.87	57.0	S.
Do	15.36	110	17.3	4.72	4.80	5.03	.98	.94	62.0	S.
Mean							.98	.846	63.7	

Vertical velocity measurements on rivers in the Catskill Mountains—Continued.

WAPPINGER CREEK, WAPPINGER FALLS, N. Y.

Date.	Gage height.	Distance from initial point.	Depth at measuring point.	Velocity in feet per second by following methods:			Coefficients for reducing to mean velocity:		Depth of thread of mean velocity (in percent of depth).	Character of bed of stream.
				Vertical velocity.	Six-tenths.	Top.	Six-tenths.	Top.		
	Feet.	*Feet.*	*Feet.*						*Feet.*	
July 7, 1903	1.70	55	2.5	1.84	1.82	2.03	1.01	0.91	60.0	S
Do	1.70	15	2.8	.96	.93	1.34	1.03	.72	59.0	S
Do	1.70	20	2.8	1.24	1.19	1.66	1.01	.75	58.0	S
Do	1.70	25	3.0	1.39	1.38	1.86	1.00	.75	60.0	S
Do	1.70	10	3.2	.58	.52	1.02	1.11	.57	52.0	S
Do	1.70	30	3.2	1.46	1.46	1.89	1.00	.77	60.0	S
Do	1.70	35	3.4	1.69	1.67	2.05	1.01	.82	59.0	S
Do	1.70	40	3.4	1.36	1.42	1.42	.96	.96	65.0	S
September 3, 1903	2.35	20	3.5	1.76	1.74	2.17	1.01	.81	59.0	S
July 7, 1903	1.70	70	3.7	1.19	1.08	1.84	1.10	.65	55.0	S
Do	1.70	80	3.7	1.33	1.25	1.84	1.06	.72	54.0	S
Do	1.70	60	3.8	1.61	1.58	1.96	1.01	.82	58.0	S
September 3, 1903	2.35	30	3.8	2.11	2.03	2.57	1.04	.82	55.0	S
July 7, 1903	1.70	75	4.0	1.52	1.52	1.81	1.00	.84	60.0	S
September 3, 1903	2.35	60	4.5	1.97	1.93	2.43	1.02	.81	58.0	S
Do	2.35	80	4.5	1.67	1.59	2.29	1.05	.73	55.0	S
July 7, 1903	1.70	65	4.7	1.24	1.17	1.66	1.06	.75	55.0	S
Mean							1.03	.777	57.8	

SUMMARY.

[Legend: Sm=smooth, R=rough, M=medium, Sh=shallow, D=deep.]

Limiting depths.	River.	Location.	Depth of mean.	Coefficient to reduce $\frac{6}{10}$ to mean.[a]	Coefficient to reduce surface to mean.[b]	Number of vertical velocity measurements.	Character of bed of river	
Feet.								
2–7	Catskill Creek	South Cairo, N. Y.	0.577	1.020	0.830	47	Sm.	M
2–5	Delaware River (east branch).	Hancock, N. Y.	.590	1.023	.816	6	R.	Sh
1.5–4	Delaware River (west branch).	.....do	.580	1.018	.831	12	R.	Sh
2–14	Esopus Creek	Kingston, N. Y.	.617	.999	.875	53	M.	D
1–5	.....do	Olivebridge, N. Y.	.577	1.012	.894	68	Sm.	Sh
2–5	Fishkill Creek	Glenham, N. Y.	.587	1.036	.790	22	M.	Sh
2–6	Hoosatonic River	Gaylordsville, Conn	.585	1.010	.830	36	M.	M
4–8	Rondout Creek	Rosendale, N. Y.	.614	.987	.840	21	R.	M
4–6	Schoharie Creek	Prattsville, N. Y.	.583	1.014	.797	11	Sm.	M
2–7	Tenmile River	Dover Plains, N. Y.	.609	.992	.884	33	M.	M
3–17	Wallkill River	Newpaltz, N. Y.	.637	.980	.846	52	Sm.	D
2–5	Wappinger Creek	Wappinger Falls, N. Y.	.578	1.030	.777	17	Sm.	Sh
	Mean		.595	1.010	.834	378		

[a] From the curve.
[b] Meter held as near the surface as possible, and not affected by wind, etc.

MEASUREMENTS ON ICE-COVERED RIVERS.

It was found necessary by T. H. Tillinghast and W. W. Schlecht, hydrographers, in computing the discharges of the rivers in the Catskill Mountains in the winters of 1901–2 and 1902–3, to study their velocities under ice and (1) to ascertain the effect of the ice covering upon the vertical distribution of velocities, especially upon the position

of the point of mean velocity, (2) to discover its effect upon the discharge of the stream, and (3) to learn how to adjust the gage heights and to compute the data obtained under ice, so as to obtain the most accurate results.

Only three of the streams under consideration remain frozen for any great length of time; these are the Wallkill, the Esopus, and the Rondout. The Wallkill is a deep river, with easy slopes and smooth bed; the Rondout is of medium depth, greater slope, and rough bed; while the Esopus is shallow, with a bed partly smooth and partly rough. Of these three special studies were made.

Observations were taken with a Price current meter, which was lowered through holes cut in the ice at regular distances across the stream. Velocities were obtained in ten or more points in the vertical, and were plotted on cross-section paper, the depths as ordinates and the velocities as abscissas. The total depth at each station was measured from the under surface of the ice to the bed of the stream. The gage height was taken at the top of the ice, and the thickness of the ice in all cases recorded.

After plotting the depths and velocities, a curve was drawn through the points and divided into ten equal parts. The center velocities of each part, taken from the curve, added and divided by 10, gave the mean velocity in that section.

When the holes were cut the water rose nearly flush with the surface of the ice, showing that it was flowing under pressure. A study of the tables and curves showed that the flow of water under such conditions was somewhat similar to its flow in pipes and in closed flumes. The mean velocity occurred at two points in the vertical, the average of 83 curves being 0.11 depth and 0.71 depth; the maximum velocity occurring at 0.36 depth. The curves dragged more for shallow depths and became more concave, and consequently the error of observation increased if the meter was not held at the proper point in the vertical depth.

The reading of the gage is an important factor in computing the flow of rivers during the time they are frozen over. In these observations the gage was always read at the surface of the water as it stood in a hole cut in the ice, this being practically at the top of the ice in all cases.

It was at first thought that by subtracting the thickness of the ice from the gage readings the regular open-channel discharge rating table might be used. It was found, however, that both the friction of the water on the ice and the head on the water itself, varied greatly, the former with the quantity of tilted, broken, and anchor ice, and the latter with the thickness of the ice and the freezing and thawing of the upper stream, and thus so complicated the problem as to render the error of this method far too great. It was found to be more accurate and satisfactory to treat ice-closed rivers as a separate problem than to attempt

to adjust the thickness of ice to the gage heights and apply them to the open-channel rating tables. Accordingly special tables were computed for use on the three streams in question when ice covered.

From the summary it appears that—

(1) The maximum velocity occurs at a point between 35 per cent and 40 per cent of the total depth, as measured from the under surface of the ice; (2) the mean velocity occurs at two points in the vertical, namely, from 0.08 to 0.13, and from 0.68 to 0.74 of the depth, measured from the under surface of the ice; (3) the depth of the stream has more effect on the point of mean velocity than the slope velocity, or the roughness of bed, this being probably due to the relation of wetted perimeter to cross section; (4) the vertical velocity curve becomes more concave as the river rises, this being due to the increased head; (5) in gaging streams under ice, measurements should be made by the vertical-velocity-curve method, or by the integration method.

Vertical velocity measurements on ice-covered streams in the Catskill Mountains.

WALLKILL RIVER, NEWPALTZ, N. Y.

[By F. H. Tillinghast.]

Gage height, read to top of ice.	Date.	Distance from initial point.	Depth from under surface of ice.	Thickness of ice.	Mean velocity.	Depth of mean velocity.		Maximum velocity.	Depth of maximum velocity.	Coefficient to reduce maximum to mean.
		Feet.	*Feet.*	*Inches.*						
11.2	January 7, 1903	60	6.8	9	2.31	0.04	0.59	2.78	0.35	0.8
	Do	80	10.3	9	2.57	.08	.68	2.96	.35	
	Do	90	10.8	9	2.40	.11	.70	2.71	.35	
	Do	110	12.3	9	2.48	.15	.85	2.82	.35	
10.9	February 10, 1903	25	4.8	16	1.29	.05	.65	1.45	.45	
	Do	40	8.0	12	2.26	.25	.80	2.47	.45	
	Do	60	6.0	12	2.18	.09	.68	2.63	.35	
	Do	80	9.6	12	2.53	.09	.69	2.86	.35	
	Do	110	11.8	12	2.60	.11	.78	3.02	.45	
7.1	February 24, 1903	60	3.8	5	1.42	.08	.64	1.91	.35	
	Do	70	4.4	5	1.55	.08	.71	1.96	.35	
	Do	85	6.8	5	1.60	.10	.68	1.84	.45	
	Do	90	7.4	5	1.50	.13	.77	1.72	.45	
8.85	February 26, 1903	25	4.0	14	.59	.10	.65	.76	.35	
	Do	40	5.8	14	1.20	.07	.72	1.42	.25	
	Do	60	4.2	12	1.11	.07	.67	1.54	.35	
	Do	80	7.8	12	1.43	.10	.72	1.70	.35	
	Do	100	9.2	12	1.41	.10	.70	1.68	.35	
	Do	110	10.5	8	1.94	.30	.77	2.10	.55	
	Do	120	7.9	7	.68	.20	.78	.81	.45	
	Mean					.12	.71		.38	.8

[By W. W. Schlecht.]

	December 11, 1901	85	11.6	0.7	2.61	0.150	0.740	3.13	0.350	
	Do	75	11.5	.7	2.64	.170	.740	3.22	.350	
	Do	53	9.2	.7	2.48	.280	.860	2.75	.550	
	Do	40	9.0	.4	2.49	.140	.750	2.89	.450	
	January 21, 1902	50	3.4	1.2	.81	.090	.730	.95	.350	
	Do	80	5.8	1.0	1.01	.130	.720	1.20	.350	
	Do	90	6.6	1.0	1.01	.120	.720	1.25	.350	
	Do	100	7.8	1.0	1.08	.120	.730	1.27	.350	
	January 23, 1902	40	14.0	1.4	3.13	.170	.800	3.61	.450	
	Do	50	16.0	1.0	3.63	.170	.840	4.00	.450	
	Do	60	14.0	.8	4.02	.130	.800	4.29	.350	
	Do	70	16.0	.8	4.02	.120	.820	4.37	.350	
	Do	80	17.0	.7	3.96	.090	.750	4.31	.450	

Vertical velocity measurements on ice-covered streams in the Catskill Mountains—Cont'd.

WALLKILL RIVER, NEWPALTZ, N. Y.—Continued.

Gage height, read to top of ice.	Date.	Distance from initial point.	Depth from under surface of ice.	Thickness of ice.	Mean velocity.	Depth of mean velocity.		Maximum velocity.	Depth of maximum velocity.	Coefficient to reduce maximum to mean.
		Feet.	*Feet.*	*Inches.*						
	January 23, 1902	90	17.0	0.7	3.93	0.070	0.700	4.36	0.350	0.900
	Do	100	18.9	.8	3.89	.160	.850	4.20	.450	.920
	Do	110	19.0	.8	3.73	.160	.760	4.18	.450	.890
	January 31, 1902	40	5.8	1.0	1.52	.150	.740	1.73	.350	.880
	Do	70	7.8	1.0	2.08	.090	.680	2.44	.350	.850
	Do	80	7.8	1.0	2.14	.110	.720	2.44	.350	.870
	Do	90	9.0	1.0	2.09	.100	.690	2.46	.350	.850
	Do	100	10.3	1.2	2.24	.140	.760	2.48	.450	.900
	February 10, 1902	80	6.5	1.0	1.42	.150	.710	1.65	.350	.860
	Do	90	7.5	1.0	1.42	.090	.640	1.72	.350	.820
	February 24, 1902	90	6.0	2.0	.85	.080	.670	1.04	.250	.820
	Do	80	5.3	2.0	.84	.130	.740	1.00	.350	.840
	Do	40	3.5	2.4	.51	.110	.700	.62	.350	.820
	Mean					.131	.744		.385	.865

ESOPUS CREEK, KINGSTON, N. Y.

[By F. H. Tillinghast.]

6.60	December 11, 1902	25	2.2	1	0.57	0.11	0.68	0.87	0.45	0.66
	Do	30	3.0	1	.87	.07	.73	1.10	.35	.79
	Do	40	4.0	1	1.13	.08	.65	1.40	.35	.81
	Do	50	3.2	1	1.21	.07	.65	1.64	.35	.74
	Do	60	3.8	1	1.24	.08	.65	1.54	.35	.80
	Do	80	7.0	1	1.28	.07	.73	1.55	.25	.83
	Do	100	6.2	1	1.06	.07	.68	1.28	.35	.83
6.90	January 14, 1903	55	2.9	7	1.08	.07	.60	1.32	.25	.82
	Do	70	3.6	7	1.08	.06	.72	1.32	.25	.82
	Do	90	7.1	7	1.01	.05	.62	1.22	.25	.83
	Do	105	4.0	7	.90	.11	.73	1.17	.55	.77
7.1	February 24, 1903	60	3.8	5	1.40	.08	.66	1.91	.35	.74
	Do	70	4.4	5	1.51	.08	.70	1.96	.35	.77
	Do	85	6.8	5	1.60	.12	.67	1.84	.45	.87
	Do	90	7.4	5	1.50	.12	.72	1.72	.45	.87
	Do	100	6.1	5	1.40	.08	.66	1.68	.45	.84
	Mean					.08	.68		.36	.80

[By W. W. Schlecht.]

	December 4, 1901	90	6.8	0.05	0.89	0.200	0.78	1.10	0.550	0.810
	Do	100	5.0	.20	.95	.080	.76	1.08	.450	.880
	January 9, 1902	80	6.6	.10	1.39	.150	.75	1.58	.350	.910
	Do	90	8.0	.10	1.15	.080	.69	1.40	.250	.820
	Do	100	6.0	.10	1.20	.160	.80	1.33	.350	.900
	February 20, 1902	80	5.7	.10	.83	.110	.78	.95	.350	.870
	Do	90	6.4	.20	.83	.060	.68	1.01	.350	.820
	February 28, 1902	40	6.0	.30	2.58	.080	.65	3.08	.350	.840
	Mean					.115	.73		.375	.856

RONDOUT CREEK, ROSENDALE, N. Y.

[By F. H. Tillinghast.]

8.50	February 23, 1903	20	5.0	8	1.45	.....	0.68	1.86	0.35	0.78
7.90	February 25, 1903	20	4.4	6	.83	0.10	.65	.97	.25	.86
	Do	70	7.0	7	1.50	.05	.63	1.93	.25	.78
	Do	80	7.6	6	1.65	.06	.68	1.91	.45	.86
	Do	90	7.6	6	1.46	.11	.74	1.75	.45	.83
	Mean					.08	.68		.35	.82

Vertical velocity measurements on ice-covered streams in the Catskill Mountains—Cont'd.

RONDOUT CREEK, ROSENDALE, N. Y.—Continued.

[By W. W. Schlecht.]

Gage height, read to top of ice.	Date.	Distance from initial point.	Depth from under surface of ice.	Thickness of ice.	Mean velocity.	Depth of mean velocity.		Maximum velocity.	Depth of maximum velocity.	Coefficient to reduce maximum to mean.
		Feet.	*Feet.*	*Inches.*						
	December 6, 1901	80	6.1	0.1	0.66	0.13	0.690	0.76	0.35	0.[illegible]
	Do	90	6.4	.2	.58	.11	.630	.68	.25	.[illegible]
	January 14, 1902	80	6.8	.2	1.24	.22	.810	1.47	.45	.[illegible]
	Do	90	6.8	.2	1.14	.16	.720	1.27	.35	.[illegible]
	February 26, 1902	92	6.2	1.5	1.29	.13	.720	1.58	.35	.[illegible]
	Do	92	6.6	1.4	1.64	.05	.670	2.02	.35	.[illegible]
	Do	80	6.4	1.5	1.51	.09	.700	1.68	.35	.[illegible]
	Do	40	5.4	1.0	1.51	.15	.750	1.67	.35	.[illegible]
	Mean					.13	.711		.35	.[illegible]

SUMMARY.

Stream.	Depth from under surface of ice.	Number of measurements.	Depth of mean velocity.		Maximum velocity.	Depth of maximum velocity.	Coefficient to reduce maximum to mean.
	Feet.						
Wallkill[a]	4 to 12	20	0.12	0.71		0.38	0.85
Wallkill[b]	4 to 19	26	.13	.74		.38	.86
Esopus[a]	2.2 to 7.4	16	.08	.68		.36	.80
Esopus[b]	5 to 8	8	.11	.73		.37	.85
Rondout[a]	4 to 8	5	.08	.68		.35	.82
Rondout[b]	5 to 7	8	.13	.71		.35	.86
Mean			.11	.71		.36	.84

[a] By F. H. Tillinghast. [b] By W. W. Schlecht.

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Murphy, Edward Charles.

. . . Accuracy of stream measurements, 2d enl. ed., by Edward Charles Murphy. Washington, Gov't print. off., 1904.

169 p., 1 l. illus., 6 pl., diagrs. 23½cm. (U. S. Geological survey. Water-supply and irrigation paper no. 95.)

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U. S. Geological survey.

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Reference.

U. S. Dept. of the Interior.

see also

U. S. Geological survey.

Water-Supply and Irrigation Paper No. 96 Series F, General Hydrographic Investigations, 11

DEPARTMENT OF THE INTERIOR
UNITED STATES GEOLOGICAL SURVEY
CHARLES D. WALCOTT, DIRECTOR

DESTRUCTIVE FLOODS IN THE UNITED STATES IN 1903

BY

E. C. MURPHY

WASHINGTON
GOVERNMENT PRINTING OFFICE
1904

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, D. C., January 9, 1904.

SIR: I have the honor to transmit herewith a manuscript prepared by E. C. Murphy, entitled "Destructive Floods in the United States in 1903," and request that it be published as a water-supply paper.

This paper treats of three of the destructive floods that occurred in the United States in 1903.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

DESTRUCTIVE FLOODS IN THE UNITED STATES IN 1903.[a]

By E. C. MURPHY.

INTRODUCTION.

Experience has shown that in the area of the United States great variations from normal climatic conditions must be expected; the temperature, or the wind velocity, or the precipitation, or all three combined, may vary from the normal by a considerable amount at any place and in any season of the year.

The year 1903 will long be remembered for its extreme local variations from normal climatic conditions. A cloud-burst at Heppner, Oreg., on June 14, caused the loss of 200 lives and a quarter of a million dollars' worth of property; a tornado and an excessive rainfall at Gainesville, Ga., caused a loss of 100 lives and of property valued at half a million dollars; the excessive rainfall of June 6 in South Carolina devastated Pacolet, Clifton, and Glendale, causing the loss of many lives and of property valued at $3,000,000, and tornadoes and excessive rainfall of the upper-central Mississippi Valley and lower Missouri Valley from May 16 to 30 resulted in the destruction of much property.

Nearly coincident in time with these extremes was the other extreme of small precipitation, with cold, backward spring, retarded growth of vegetation, forest fires, and very small flow of streams in New York and the New England States.

THE HEPPNER, OREG., FLOOD.

The flood of Willow Creek, Morrow County, Oreg., commonly spoken of as the "Heppner disaster," occurred on Sunday evening, June 14. It was due to one of those very heavy storms of short duration covering a very small area, so peculiar to the arid region, and popularly called a "cloud-burst." Such a storm is almost like the tornado in its suddenness, destructibility, and limited extent. The duration of this storm was only half an hour, and the resulting flood lasted

[a]The writer desires herewith to express his thanks to Mr. F. H. Newell, hydrographer in charge, and Mr. J. C. Hoyt, computer, for valuable suggestions and assistance in the preparation of this paper, and to Messrs. J. T. Whistler, B. S. Drane, and W. G. Russell for their reports on the floods in their respective districts.

less than one hour, yet in that short time one-third of the town of Heppner was entirely swept away, one-quarter of a million dollars' worth of property destroyed, and over 200 persons, from a total population of 1,400, were drowned.

Character of stream and country.—Willow Creek rises in the southeastern part of Morrow County in the west spur of the Blue Mountains, and flows northwest into Columbia River. The distance in a straight line from its source to its mouth is about 100 miles. Its three principal tributaries are Balm Creek, which enters a short distance above Heppner; Hinton Creek, which enters a short distance below Heppner; and Wilson Creek, the mouth of which is about 25 miles below Heppner. At its mouth Willow Creek has an elevation of about 220 feet, and at Heppner, 45 miles by rail above its mouth, it is 1,920 feet above sea level. The mean slope is, therefore, 37.8 feet per mile. The channel at Heppner has a width of 30 to 40 feet, and the banks are from 10 to 15 feet in height. The valley is a fertile strip of cultivated land, 500 to 1,500 feet in width. Back from the valley the country on both sides has barren rocky soil, with steep slopes, and the precipitation passes off very rapidly.

Sources of information.—Unfortunately there were no measurements of rainfall and gage heights in the watershed of Willow Creek, and the extent of the storm and the volume of run-off can not be determined. All the data on which are based the estimates of run-off, duration of storm, and height and duration of flood were obtained from persons who saw the flood at various points along Willow Creek.

On the fourth day after the flood Mr. John T. Whistler, engineer of the United States Geological Survey, visited Heppner and made a thorough study of the situation. From his description, published in Engineering News of July 16, and from additional facts obtained from personal conversation with him, this report has been prepared.

The storm.—The storm came on suddenly between 4 and 5 p. m., and had entirely passed away in less than half an hour. Authentic accounts agree that it started in the foothills of the Blue Mountains, on Balm Creek, about 8 or 10 miles north of Heppner, and moved north, down Balm Creek, to some distance beyond Willow Creek. It is estimated that the storm area was from 2 to 4 miles in width and from 8 to 10 miles in length and affected an area of perhaps 20 square miles.

The storm was accompanied by a very heavy fall of hail, the depth of which can not definitely be ascertained, but an idea of its amount can be formed from the fact that five days after the storm some bodies were found in drifts of hail in a nearly perfect state of preservation. Some of the hailstones are said to have measured 1¼ inches in diameter. Five days after the storm some that measured five-eighths of an inch by seven-sixteenths of an inch were removed from a house buried under silt and brush. Nearly all of this hail was of clear ice.

unlike the usual hailstones, which are more or less opaque and built up from a nucleus by successive layers.

The flood.—The height of the flood may easily be determined from the high-water marks, though these are often deceiving to the unpracticed eye, as marks are frequently found in places where the water did not reach. Bark is knocked off trees by floating bodies that project a considerable distance out of the water; small trees are often bent over and light drift deposited on them, and when the water pressure is removed and the tree straightens up this material is lifted high above the water level. But the rings of silt left on trees, all at approximately the same level, show the true water line. These watermarks show that the flood plane at Heppner was about 5 feet above the general road level and had a width of about 500 feet. The maximum depth of the flood at the center of the creek was from 15 to 20 feet.

The crest of the flood wave is said to have been almost simultaneous with the first water as far down the creek as Lexington; so that if a person had been standing on a structure 40 feet in height above the center of the creek bed at Heppner he would have seen flowing down the creek a bank of water 20 to 25 feet in height and having a face slope of about 30° with the horizontal. The force of a bank of water of this height moving down a slope of 38 feet to the mile, sweeping everything before it, can readily be imagined. At Ione, 8 miles below Lexington, the crest of flood was one-half hour behind first water, and at Cecils, 14 miles above the mouth, it was several hours behind.

At Heppner the creek is said to have returned to its normal flow in one and one-half hours after the passage of the crest of the flood. The crest is said to have reached the mouth of the creek about twenty hours after passing Heppner, the speed of crest thus averaging from 5 to 6 miles an hour.

The discharge on Balm Creek near its mouth was computed from the formula:

$$v = C\sqrt{s\,r}$$

s being the slope, r the hydraulic depth, C a coefficient, and v the mean velocity.

Two sections 2,200 feet apart gave a slope of 0.025; area, 1,700 square feet; wet perimeter, 260 feet. With these data and the value of 0.040 for the coefficient of roughness, the maximum discharge is found to be 36,000 second-feet. As the flood lasted less than one hour, the total flood volume was probably not more than the maximum flow for one-third of an hour. On this assumption the total volume of the flood at this point amounted to about 1,000 acre-feet.

The discharge of Willow Creek at Heppner and at Lexington has been computed. In these computations the mean velocity was assumed to be 6 miles per hour at Heppner and 4 miles per hour at Lexington. The total volume of flow was also assumed to be the maximum flow

for one-third of the time. The total flood volume at Heppner was found to be 1,100 acre-feet, and at Lexington 800 acre-feet.

The increased discharge at Heppner over that in Balm Creek was due principally to the water which enters Willow Creek below the mouth of Balm Creek, but partly to a small amount from shore. The flood volume near the mouth of Willow Creek was also computed and found to be 750 acre-feet.

It is very likely that the storm area was at least 20 square miles. Assuming that two-thirds of the rainfall reached Heppner, the total precipitation was approximately 1,600 acre-feet, which amounts to 1½ inches in depth over this area, a rate of rainfall of about 3 inches per hour.

The flood water was very heavily laden with silt and in color is said to have resembled molasses.

Effect of the flood.—As a result of the flood one-third of the town of Heppner was entirely destroyed, and about 200 lives were lost. It seems almost incredible that a flood of a depth of only 5 feet above the general road level of the town should cause such a loss of life. The explanation lies in the peculiar construction of the houses. Nearly all of the houses simply rested on posts or open foundations of stone. The water freely passed underneath, but could not so rapidly get to the inside of the houses, so they were lifted off their foundations and floated away like boats. The rapid velocity soon drove them against trees or other objects and crushed them to pieces. In such a swift current and among such masses of timber it was almost impossible for a person to get ashore.

The duration of the flood was so short that the only considerable scouring effect observed was at the mouth of Balm Creek, where the flood crossed Willow Creek and struck the opposite shore. At this point a new channel was formed.

In 1888 a similar flood, possibly a larger one, came down Hinton Creek, but did little damage, as there were only a few houses there at that time. In 1883 there was a large flood in Willow Creek, but reliable records are not available for the comparison of these floods with the one of 1903.

Lessons taught by the flood.—Storms of great violence, short duration, and affecting comparatively small areas are to be expected occasionally in all parts of the arid and semiarid region, as the bare, steep, nonabsorbent surface causes the resulting floods to be large, sudden, and very destructive. All streams should have an ample waterway under bridges and through towns, and all buildings should be out of reach of the flood. No building that can be lifted from its foundation and swept away should be allowed in the area of a possible flood. Buildings in the line of a flood, such as mills, warehouses, pumping stations, etc., should be protected from drift during high water.

THE SOUTH CAROLINA FLOOD.[a]

A destructive flood occurred in Spartanburg County, S. C., on June 6, 1903. Considering the small area affected by the storm, the precipitation was very large and the loss of life and property great.

Character of country and streams.—In the twenty-four hours of the storm there was a rainfall of from 3½ to 5 inches in the counties of Cherokee, Spartanburg, Greenville, and the eastern half of Pickens, in South Carolina; and in the southern part of Rutherford, Polk, and Henderson counties, in North Carolina—a total area of 2,300 to 2,500 square miles. This region is situated on the southern slope of the Saluda Mountains. It does not extend up into the mountains, but includes the foothills and rolling country. About half of it is covered with timber; the remainder is cultivated and pasture land. The surface slopes are such that the water runs off rapidly and there is very little storage.

The streams that drain this area are Pacolet, North, South, and Middle Tiger, and Enoree rivers. These are tributaries of Broad River and enter it from the west. The upper tributaries of Saluda River, which flow to the west, rose to an unusual height, but little damage was done on them. Practically all the damage was confined to Spartanburg County, S. C.

The streams mentioned rise within a comparatively short distance of each other. North Fork of the Pacolet heads in Polk County, N. C.; all the others head in Greenville County, S. C. They are characterized by deep, narrow channels, with rapid fall. The Pacolet is said to have a fall of 500 feet from its head to Pacolet mills, a distance of 35 miles. There are no ponds or wide bottom lands to hold back the water in time of flood. The ground was soaked almost to saturation just previous to the flood, so that there was very little ground storage.

The storm.—During the week preceding this flood there was rain over this area almost daily; not hard rains, but gentle, penetrating showers, that nearly saturated the soil. On Friday, June 5, 1903, rains fell almost incessantly. In the evening the rain became more violent and continued until 4 a. m. without interruption.

[a] Prepared from report of B. S. Drane, assistant hydrographer.

The precipitation for the first six days of June, as reported to the United States Weather Bureau, is given in the following table:

Daily rainfall reported from Weather Bureau stations in and surrounding area of South Carolina flood for the first six days in June, 1903.

Station.	June 1.	June 2.	June 3.	June 4.	June 5.	June 6.
	Inches.	*Inches.*	*Inches.*	*Inches.*	*Inches.*	*Inches.*
Waynesville, N. C	0.30	.....	0.10	0.40	0.80	0.[illegible]
Asheville, N. C	.59	0.10	.....	.06	.99	.6[illegible]
Morganton, N. C	.62	.86	.14	1.50	.....	1.1[illegible]
Statesville, N. C	.35	.54	.....	.....	.70	1.2[illegible]
Hendersonville, N. C	.61	.74	.40	.....	1.22	1.[illegible]
Brevard, N. C	1.60	.40	.....	.30	2.00	2.1[illegible]
Highlands, N. C	1.10	1.57	.....	1.28	3.20	1.2[illegible]
Henrietta, N. C	.19	1.44	.....	Trace.	1.51	4.[illegible]
Charlotte, N. C	Trace.	.51	.07	.10	.59	1.5[illegible]
Liberty, S. C	1.20	Trace.	.....	.25	.55	2.0[illegible]
Spartanburg, S. C	Trace.	.90	.07	.....	.35	5.0[illegible]
Greenville, S. C	.27	.80	.08	.....	1.15	3.[illegible]
Gaffney, S. C	.....	.10	.82	.09	.94	3.[illegible]
Santuc, S. C	.....	1.15	.....	.73	.87	.[illegible]
Rock Hill, S. C	.04	1.17	.02	.33	1.15	1.[illegible]
Columbia, S. C	.82	2.30	.12	.41	.22	2.[illegible]
Heath Springs, S. C	.80	1.10	.01	1.50	.60	1.4[illegible]
Barksdale, S. C	1.80	.60	.....	.....	2.75	1.[illegible]

Fig. 1 shows the location of some of these stations and the depth of rainfall reported at each on the morning of June 6, 1903. It is seen that the greatest precipitation in the twenty-four hours was at Spartanburg, diminishing in amount in every direction from that place, rapidly to the northwest and southeast and gradually to the northeast and southwest.

The precipitation northwest of Spartanburg was considerably greater than 5 inches. The washouts, landslides, and gullies cut by the flowing water indicate that the precipitation over the rain area was probably from 7 to 11 inches in the twenty-four hours preceding the morning of June 6, 1903. This region is included in an area that Prof. A. J. Henry, of the United States Weather Bureau, states is second in precipitation in the United States. The people who live in that region are accustomed to very heavy rains, and their estimates confirm the figures given above of the precipitation.

A. VIEW OF DAMAGED PORTION OF PACOLET MILL, NO. 3.

B. VIEW OF WRECKAGE OF PACOLET MILLS, NOS. 2 AND 3.

The flood.—There are no gaging stations on the streams on which the damage was done by this flood. For about seven years the United States Geological Survey has maintained gaging stations on Broad

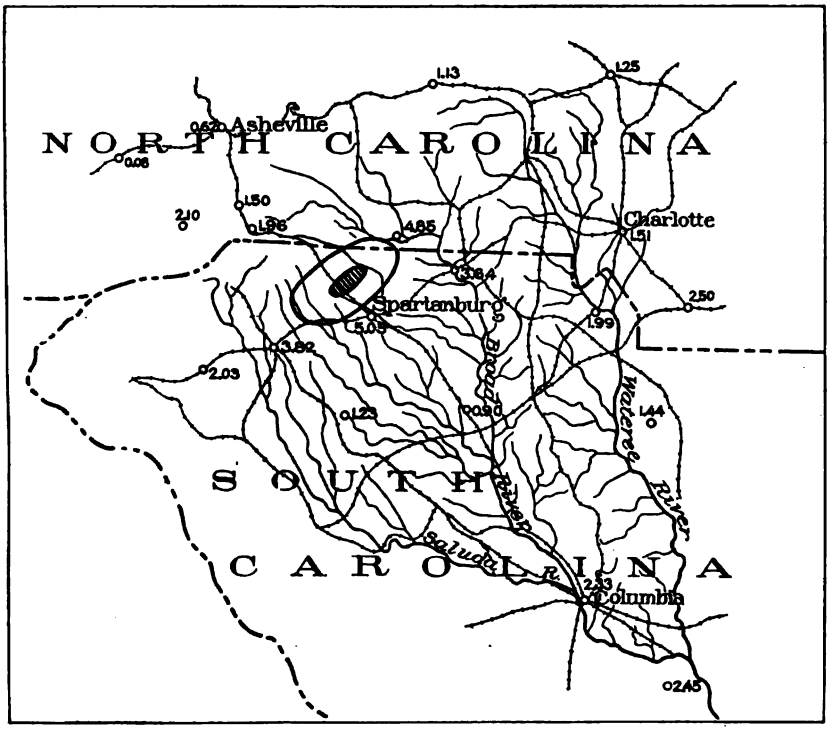


FIG. 1.—Map showing distribution of Weather Bureau stations, etc.

River at Alston, about 75 miles below Pacolet, and on the Saluda near Waterloo. The mean daily gage heights at each of these places from June 1 to 10 are given in the following table:

Mean daily gage heights, in feet, June 1 to 10, 1903.

Place.	1.	2.	3.	4.	5.	6.	7.	8.	9.	10.
Broad River at Alston, S. C ...	5.60	7.55	8.70	6.65	7.30	13.60	27.20	26.70	17.30	9.90
Saluda River at Waterloo, S. C.	9.85	10.40	8.40	7.75	12.15	14	19.25	22.35	14.45	10.30

The highest water previous to this flood at these stations during these seven years was in 1901. At Alston, S. C., the maximum on June 7, 1903, was 29 feet, which is 5½ feet higher than in 1901. At Waterloo the maximum was 23.15 feet, which is 2¼ feet higher than in 1901.

These figures, while they convey some idea of the magnitude of this flood, are not definite, because these stations are far below where the

storm occurred and where the damage was done. Had there been a gaging station on Pacolet River near its mouth it would have shown a much greater difference between the flood readings of 1901 and 1903 than do the figures at the Alston station.

A few flood marks of the flood of 1852 still remain at points on these streams. At Fingerville the new mark is 3.7 feet higher than the old one. At Whitneys mill, on Lawsons Fork near Spartanburg, the new mark is 9.9 feet higher than the old one. On the main Pacolet and Middle Tiger rivers the new marks are about 10 feet higher than the old ones. On Enoree River the new mark is 1½ feet lower than the one of 1852.

For computing the discharge a section was taken above the dam at Pacolet. The pond at this point is shallow and the section rather narrow, and it is therefore believed that the current was fairly uniform. On the right bank the high-water mark was found in a garden, and is undoubtedly reliable. This gives a difference in elevation between high and low water of 24.4 feet and a flood section of 6,150 square feet. At gaging stations during floods, when the section was similar and the slope similar or not so great, the mean velocity has been observed to be as much as 5.78 feet per second. Assuming this velocity and using the area of section given above, the discharge at this place is 35,547 second-feet. The drainage area of the Pacolet at this place is about 400 square miles. The estimated discharge is equivalent to a run-off of 88.9 second-feet per square mile.

The flood of Catawba River of May, 1901, which caused great destruction of farming land, had an estimated run-off per square mile of 51.8 second-feet at the Catawba, N. C., station, and of 44.4 second-feet at the Rockhill, S. C., station. The estimated run-off at Pacolet during the 1903 flood is seen to exceed that of 1901 at the Catawba by 70 per cent and at Rockhill by 100 per cent. It is possible, however, that the estimated run-off at Pacolet is too great. The method of obtaining the mean velocity was not very reliable.

Destruction wrought by the flood.—There was some destruction of property on Tiger River, but the largest amount was on Pacolet River, at Clifton mills and Pacolet mills. The destruction of the Clifton mills is thus described in the report of Mr. B. S. Drane:

> Clifton mill No. 3, known as "Converse mill," stood just above the bridge over which the main line of the Southern Railway crosses Pacolet River. It was built in 1890 and 1895, and as completed was a four-story mill 496 feet long and 100 feet wide. It was valued at $1,000,000 and contained 50,100 spindles.
>
> The river was noticed to be about 8 feet above low water at midnight, but did not appear to be rising, and caused no anxiety. At 3.30 a. m. Mr. Kirby, master machinist of the mill, upon going down, saw that the water was beginning to rise into the wheelhouse. Getting assistance, he began to remove belts and ropes, but before any headway had been made the water forced him to leave. By 4.30 the rise was beginning to be rapid, and in a few minutes the water was flowing over the right end of the dam and breaking on the boiler houses and shops. These were wrecked before 5 o'clock. A little after 5 the brick stack, 137 feet high, was washed down with the part of the dam immediately behind it; it fell

A. PACOLET BEFORE THE FLOOD OF 1903.

B. PACOLET DURING THE FLOOD OF 1903.

on the corner of the mill. The mill began to wash down immediately, and the wreck was practically complete by 6 o'clock. The mill was washed entirely away, except for the three-story picker room. Shops, boiler houses, wheelhouse, and 16 cottages were also carried away.

The dam here had a section of 27 feet height, 27 feet base, and 11 feet crest. The construction was said to have been hasty and of materials much inferior to those usual on this river. The crest was broken off irregularly to a depth averaging 10 feet for 100 feet from the right bank, and to a depth of 7 feet for the next 40 feet. One footbridge and one county bridge, of wood, were carried away from just below the mill.

About 1½ miles below was the original Clifton mill, known as "No. 1." It was built in 1880, and had 27,500 spindles. At this point the rise was not alarming at 2 o'clock a. m. The rapid rise began at some time between half past 4 and 5. Just after 5 the water rose 2 feet in ten minutes, according to the assistant superintendent. The first thing to be carried away was a large hall. This floated off at about 5.45 and was hurled down into the upper corner of the mill, which stood out into the stream. The blow broke in part of the corner of the mill, but the destruction of the mill did not begin until the wreckage from above began to come down thickly, a little after 6 o'clock. Large timbers were then dashed into the projecting corner and the mill began to settle down gradually, little by little, beginning at this end. Altogether about 110 feet of the main building and the wheelhouse were totally wrecked. The machinery on the two lower floors was severely damaged throughout the length of the mill by the water, mud, and drift. The dam was uninjured, except for the tearing off of the wooden apron. Fifteen cottages were carried away from this mill village. About 8 o'clock the water began very slowly to fall.

Clifton mill No. 2, known as "Dexter mill," was a little over a mile from Clifton, was built in 1888, and had 30,500 spindles.

This mill stood on a point on the left bank. Just opposite the mill the right bank is steep and high, while on the left bank a ridge extends out on the point almost to the mill. Above the mill, on either side of the river, the hills open out, and there the greater part of the mill village stood. Also on the point on the left bank below the mill were many cottages.

The rise here began to alarm the watchman at 3 a. m., and at 3.30 it began to be rapid. One of the watchmen noticed that the rise was 1 foot in five minutes at about 4 o'clock. The alarm was sent out among the operatives at once. Most of them refused to realize that they were in any danger and would not make any effort to save themselves and their property until the water surrounded them. There were many narrow escapes, but at least 52 persons were lost here, most of them women and children. There was a deep deposit of sand and silt over the point below the mill where most of the lives were lost. Toward the end of the week following the disaster this began to crack as it dried. Searching parties kept watch of these cracks, and wherever flies were seen to crawl down them they dug. Several human bodies were found in this way.

At about twenty minutes past 4 the covered wooden highway bridge was washed down. The cotton warehouses and some of the cottages began to go about 6 o'clock, but the mill stood until 7. At that time the wreckage was coming down thickly from the upper mills. Exactly as at the Clifton mill above, the outer upstream corner was torn down. This mill was L-shaped, a wing extending shoreward from the lower end. When the mill was surrounded by the flood the heavy wreckage from above was hurled into this extension, tearing down the middle of it and practically filling the whole with driftwood. The machinery throughout the two lower floors of this mill was largely destroyed by the sand and drift thrown into it. All the cotton and cloth warehouses here were washed

away; 42 of the company's cottages and 6 belonging to outsiders were wrecked and carried away.

After bending to the left about the point on which the mill stands the river formerly bent to the right again around a low point on the right bank. During this flood the water overflowed this low point and cut a new channel over it, abandoning the old one. The crescent-shaped island left is about 400 yards long and 100 wide.

Several lines of the Southern Railway were tied up for about a week. The main line from Charlotte, N. C., to Atlanta, Ga., passing through Spartanburg, was broken by the washing away of a bridge over Lawsons Fork of Pacolet River. The Charleston and Western Carolina Railway, also entering Spartanburg, lost its bridge over Tiger River.

What purports to be a very conservative estimate of the financial loss due to the freshet is given as follows:

Estimate of losses caused by South Carolina flood.

Clifton mills, Nos. 1, 2, and 3	$1,750,000
Pacolet mills, Nos. 1, 2, and 3	1,250,000
Glendale mills	30,000
Mary Louise mills	18,000
Whitney mills	5,000
Fingerville mills	8,000
Tucapau mills	10,000
Tiger mills	8,000
Two other mills	20,000
Bridges	15,000
Spartanburg Electric Railway Company	30,000
Southern Railway Company	300,000
Not included above (lands and crops ruined, etc.), at least	6,000
Total	3,450,000

These figures do not include the loss due to the interruption of railway traffic, the stoppage of large industries throwing thousands of operatives out of employment, the loss to the various trades and commercial enterprises, and, worst of all, the loss of over 50 lives.

Lessons taught by the flood.—Very heavy storms, covering small or large areas, are to be expected on the southern and eastern slopes of the Appalachian Mountains. Floods much greater and more destructive than ordinary result from these storms.

Ample unobstructed waterway must be provided for streams through towns. Usually the greater part of the destruction wrought is due to the failure of a temporary dam formed by drift in front of a bridge. Drift of all kinds collecting in front of a bridge forms a dam, raises the water several feet higher than it would have been without the obstruction, finally the pressure on the obstruction becomes so great that it is swept away, and the resulting wave destroys other bridges and buildings that otherwise would not have been injured. The washing out of a dam by the water cutting a channel around one end of it

A. CLIFTON BEFORE THE FLOOD OF 1903.

B. CLIFTON AFTER THE FLOOD OF 1903.

is often the cause of a great wave that sweeps away bridges and buildings below that otherwise would not have been injured.

Buildings that can be lifted from their foundations and swept away should not be allowed in the path of a possible flood.

Mills, power stations, and other buildings on the banks of streams should be protected from the destructive action of drift during high water.

NOTES ON EARLY FLOODS IN MISSISSIPPI VALLEY.

Data on the floods of the Mississippi River basin prior to 1798 are very meager. Since that date a record of the fluctuation of the surface has been kept at one or more points along the river. At Natchez a record was kept by Governor Winthrop Sargeant from 1798 to 1819, by Samuel Davis from 1819 to 1841, and by Professor Forshey from 1841 to 1848. From 1848 to 1851 a record was kept at Carrollton by Professor Forshey. From 1851 to 1860 a very comprehensive study of the lower basin was made by Humphrey and Abbott. From 1851 to the present time several gaging stations have been maintained by the United States engineers, and for several years the United States Weather Bureau has also measured surface fluctuations at several places in this basin. From the available data the following tables have been prepared.

The following table,[a] from the records of Dr. George Englemann, shows the date of each flood on the Mississippi River at St. Louis from 1785 to 1865 and the height of the highest water above a datum plane:

Statistics of floods at St. Louis, Mo., 1785 to 1865.

Date.	High water.	Date.	High water.
	Feet.		*Feet.*
April, 1785	42.0	May 20, 1852	28.0
March, 1828	33.6	May 7, 1853	30.0
May 8, 1838	27.0	May 9, 1856	27.4
May 28, 1838	27.4	June 14, 1858	37.0
May 2, 1843	27.2	May 15, 1861	25.3
June 27, 1844	41.0	April 26, 1862	31.3
June 27, 1845	32.4	May 4, 1863	18.0
May 10, 1846	25.0	May 14, 1864	24.3
March 10, 1849	27.4	July 28, 1865	26.7
June 10, 1851	27.0		

According to these records the highest water at St. Louis, Mo., was in April, 1785, when the gage read 1 foot higher than in 1844 and 4 feet higher than in June, 1903. Very little is known of the flood of

[a] Russell, T. Stages of Mississippi River and its principal tributaries. U. S. Weather Bureau, Washington, D. C., 1891.

1785. The 1844 flood is well remembered by persons still living in this section of the country. In 1858 high water at St. Louis came within 4 feet of that of 1844, and the flood of 1845 was within about 9 feet of that of 1844. The following table gives the dates of highest water on the Mississippi at St. Louis, Mo., and on the Missouri at Kansas City, Mo., for each year from 1872 to 1903; also the gage reading:

Statistics of floods at St. Louis, Mo., and Kansas City, Mo.

Year.	St. Louis, Mo.		Kansas City, Mo.	
	Date.	Gage height.	Date.	Gage height.
		Feet.		*Feet.*
1872	June 12–14	23.7		
1873	April 11	26.7	July 5	19.3
1874	June 19–20	19.1	June 17–19	16.2
1875	August 3	30.5	April 30	17.8
1876	May 10	32.7	June 16	18.0
1877	June 14	27.3	June 10	22.2
1878	June 15	26.5	July 3	19.8
1879	July 3	21.9	June 30	14.2
1880	July 2	26.2	July 10-12	16.7
1881	May 6	34.3	April 30	26.3
1882	July 5	33.1	July 3	19.2
1883[a]	June 26	35.5	June 26	24.8
1884	April 9–10	28.8		
1885	June 17	27.8	June 19	19.1
1886	May 13	27.7	April 17	15.8
1887	April 3	21.4	July 3	18.9
1888	June 4	30.1	do	20.4
1889	June 1	25.3	August 14	13.9
1890	do	21.3	June 11	17.2
1891	July 4	24.4	July 1	23.1
1892	May 19	36.7	May 21	24.9
1893	May 3	32.3	April 13	17.7
1894	May 11	24.1	June 21	20.1
1895	July 8	17.8	June 12	16.9
1896	May 26–28	28.4	May 22	19.2
1897	May 2	31.7	April 20	22.6
1898	May 23	27.2	June 13	21.5
1899	April 27	25.6	July 9	22.2
1900	March 16	23.4	June 20	17.8
1901	April 18–19	22.4	June 24–25	19.4
1902	July 26	26.9	July 15	23.2
1903	June 10	38.0	June 1	35.0

[a] Record May 1 to June 16 and July 1 to December 31 missing.

It is seen that at no time during the fifty-nine years from 1844 to 1903 has the highest water been within 3 feet of the stage it reached on June 27, 1844. Seven times, however, it approached within about 8 feet of it. The records of the Kansas City station on Missouri River cover a period of thirty years; the flood of 1903 exceeded in height by about 9 feet that of any year during this period; only nine times in the thirty years was the highest water at this place within 15 feet of its height in 1903.

The years of greatest flood at St. Louis are usually years of greatest flood in the lower basin of Mississippi River, but this is not always the case. Very frequently the lower Mississippi is very high while at St. Louis the stage is considerably below that of the great flood. This is due to a large part of the discharge of the lower basin coming from Ohio and Arkansas rivers, which enter the Mississippi below St. Louis. The frequency of floods and their magnitude increases with the distance down the river. Floods are the exception at Kansas City; they are the rule below Memphis.

KANSAS FLOODS AND HYDROGRAPHIC DATA.

THE FLOOD OF 1903.

The Kansas flood of 1903 is, with the exception of the flood of 1844, the largest one on record in the upper-central Mississippi and lower Missouri river valleys. In loss of property and life the flood of 1903 was vastly more destructive than that of 1844, on account of the country now being much more thickly settled and improved. Although the loss of property in the aggregate is large—estimated at $22,000,000 in Kansas and Kansas City, Mo., alone—it is not large when one considers the great value of the property in the path of the flood. The loss of life is surprisingly small considering the population driven from their homes and the rapid approach and vastness of the flood. The following table (p. 22) gives flood data of Kansas River taken at Lawrence and Lecompton.

Flood data, Kansas River, 1881 to 1903.

Year.	Place.	Date.	Gage reading.	Discharge.
			Feet.	*Sec.-ft.*
1881	Lawrence, Kans.	Mar. 7	4.2	
1882	do	Apr. 10	4.3	
1883	do	June 25	4.3	
1884	do	May 5	3.8	
1885	do	Apr. 27	3.7	
1886	do	May 10	4.3	19,370
1887	do	May 23	2.5	8,840
1888	do	June 25	3.5	14,290
1889	do	May 13 July 22	5.0	24,340
1890	do	Jan. 13	2.8	10,370
1891	do	June 1	6.0	[a]35,598
1892	do	May 16	8.8	[a]67,718
1893	do	June 5 July 3	4.3	19,370
1894	do	June 25	5.3	26,620
1895	do	June 10 Aug. 19	4.0	17,390
1896	do	July 20	7.7	[a]53,308
1897	do	Apr. 26	8.8	[a]67,718
1898	do	June 10	5.6	28,990
1899	Lecompton, Kans.	July 8	11.0	30,250
1900	do	Mar. 10	10.2	24,898
1901	do	Apr. 14	10.0	25,000
1902	do	July 15	17.6	81,400
1903	do	May 31	29.5	233,070

[a] Computed from rating table for period May 15 to Aug. 31, 1896.

From this table it is seen that Kansas River at Lecompton was 12 feet higher during this flood than at any time during the twenty-two years covered by the record. The gage reading at this place ordinarily varies from 4 to 12 feet, but on May 31 it read 29½ feet, or 17½ feet higher than ordinary high water. The volume of water flowing in Kansas River on May 31 was nearly three times greater than at any other time during these twenty-two years.

THE STORM.

Duration, extent, and depth of rainfall.—The period of excessive rainfall dates from May 16 to 31; the temperature during this time was abnormally high, the pressure was low, and a strong wind was

blowing from the south, which developed tornadoes in parts of Iowa, Nebraska, and Kansas, killing 50 persons, injuring about 60 others, and destroying many thousands of dollars' worth of property. The daily rainfall during this period is shown in the following table, which is condensed from a table taken from the Monthly Weather Review for May, 1903:

Daily rainfall at Weather Bureau stations, May 16–31, 1903.

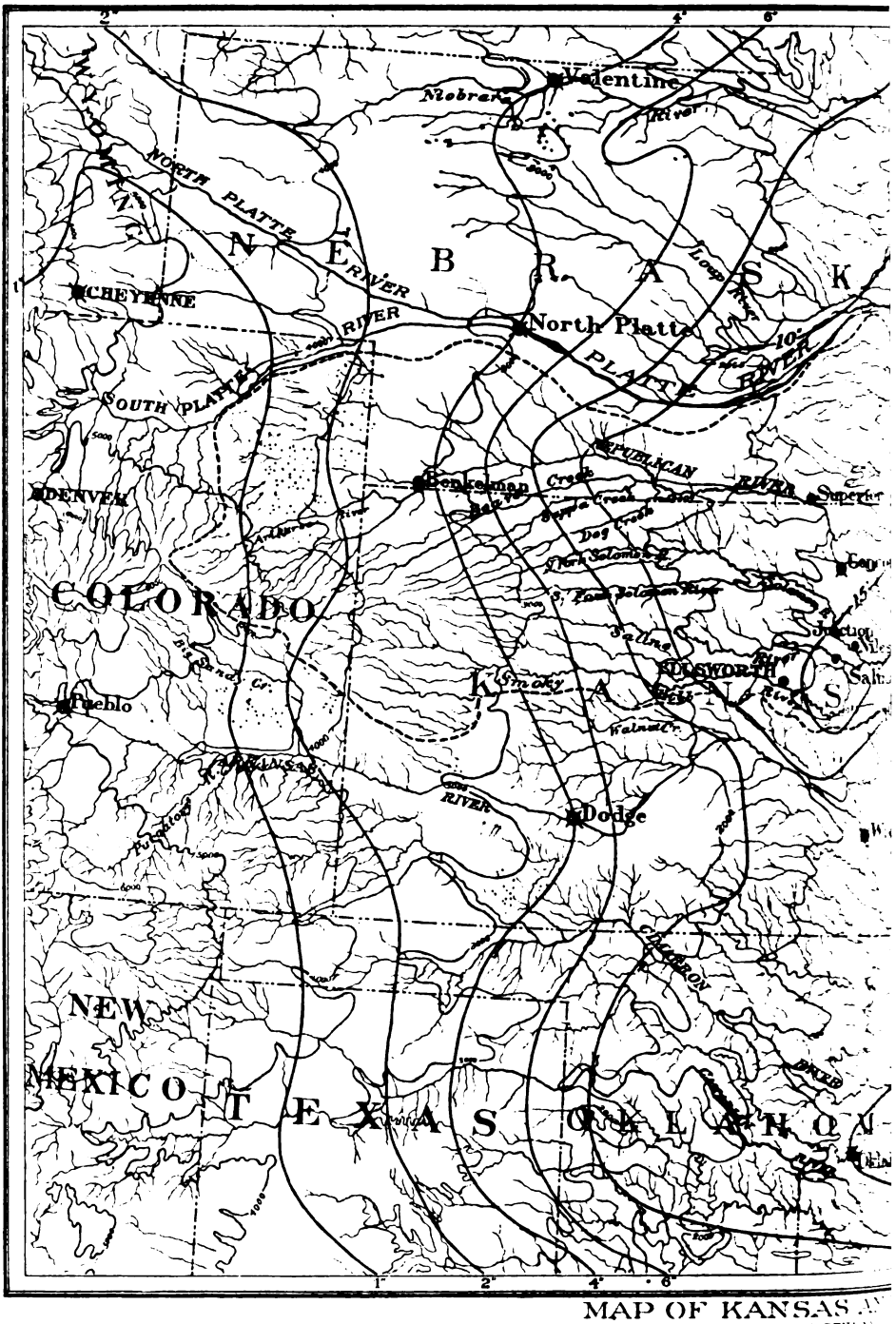
Station.	16.	17.	18.	19.	20.	21.	22.	23.	24.
Williston, N. Dak	0.16	0.08	0.34	T.	T.	0.94	1.49	0.12	0.02
Bismarck, N. Dak	.....	T.	.86	T.	.....	.47	1.06	.....	.....
Pierre, S. Dak	.....	.26	.....	.....	.....	.14	.62	.....	.19
Rapid, S. Dak	.02	.14	.30	0.16	.....	.14	.18	.....	T.
Huron, S. Dak	.....	.....	.....	T.	.....	.16	.50	.34	.24
Yankton, S. Dak	.....	.....	.....	.03	T.	.39	.76	.63	1.00
St. Paul, Minn	.....	T.	.....	.10	.....	.....	.01	.64	T.
Moorhead, Minn	.....	.04	.30	.60	.....	.24	.42	1.50	.....
Valentine, Nebr	.....	.....	.....	.....	.....	.26	.24	T.	.10
North Platte, Nebr	T.	.....	.....	.....	.....	.18	.02	.....	.02
Lincoln, Nebr	.....	.07	T.	.24	.....	.56	.79	.28	.....
Omaha, Nebr	.....	.16	.....	.04	.10	.64	.80	.01	.....
Concordia, Kans	.30	.14	.78	.04	.....	.18	1.10	.94	.....
Topeka, Kans	T.	.....	.....	.05	.....	.50	.04	.64	.35
Wichita, Kans	.01	.....	.98	.10	.....	2.24	.04	1.29	T.
Dodge, Kans	.14	.10	.....	.....	.10	.58	.06	.....	.....
Denver, Colo	T.	T.	.01	.....	.....	T.	T.	T.	.....
Pueblo, Colo	.....	.....	.....	.....	.....	.....	.....	.....	.....
Cheyenne, Wyo	.10	.06	.10	.....	.....	T.	.01	.....	T.
Oklahoma, Okla	.58	T.	.01	T.	.....	.02	1.25	T.	4.06
Fort Worth, Tex	T.	.01	.....	.....	.....	T.	.....	T.	T.
Dubuque, Iowa	.....	.08	T.	T.	.01	.06	.50	.02	.....
Davenport, Iowa	.....	.....	T.	T.	.....	.48	.08	.20	.05
Des Moines, Iowa	.....	.30	T.	.04	.02	.06	1.38	.12	.....
Keokuk, Iowa	.....	T.	.06	T.	.....	.76	.04	.06	1.40
Sioux City, Iowa	.....	.02	.72	1.08	.....	.....	.88	.24	.02
Kansas City, Mo	.....	.02	.....	.06	T.	.70	.34	.74	.26
St. Louis, Mo	.....	.....	.16	.04	.40	.06	.....	.....	T.
Springfield, Mo	T.	.04	.02	.70	1.06	.35	.....	.....	.06
Hannibal, Mo	.....	T.	.17	.02	.....	.49	.....	.04	1.93
Fort Smith, Ark	T.	.03	T.	1.36	T.	.....	T.	.....	.....
Little Rock, Ark	T.	T.	T.	.54	T.	.....	.....	.....	.....

Daily rainfall at Weather Bureau stations, May 16–31, 1903—Continued.

Station.	25.	26.	27.	28.	29.	30.	31.	Total.
Williston, N. Dak		0.08		0.60	0.04			3.87
Bismarck, N. Dak		.20					0.50	3.09
Pierre, S. Dak		.04			.46	T.		1.71
Rapid, S. Dak	0.02			T.	.12		.16	1.4
Huron, S. Dak	.08	.24	0.01					1.57
Yankton, S. Dak		.39	.39	.43	.40	0.10	.03	4.55
St. Paul, Minn	.62	1.04	.24					2.65
Moorhead, Minn								3.10
Valentine, Nebr	.04	.50			.32	.02		1.48
North Platte, Nebr	.08	.18		.01	.43	.06		.98
Lincoln, Nebr	1.37	1.03	.43	.15	1.94	1.13	.10	8.09
Omaha, Nebr	.95	.93	.56	T.	.52	.40	.10	5.21
Concordia, Kans	.16	.96		1.64	3.68	.25	.42	10.59
Topeka, Kans	.22	.21	.28	.24	1.71	.26	.91	5.42
Wichita, Kans		.22		.22	.82	.02	.26	6.17
Dodge, Kans		.30	.26	.28	.03	.04		1.89
Denver, Colo	.01	T.		.01	.03		.28	.34
Pueblo, Colo		.06		.22	T.			.28
Cheyenne, Wyo		T.		.12	T.			.39
Oklahoma, Okla		T.		.18	3.26	.02	T.	9.38
Fort Worth, Tex		T.		.68	.46	T.		1.15
Dubuque, Iowa	.10	1.40	1.11	.02	T.	.34	.12	3.76
Davenport, Iowa	.08	.87	.42	.38	.32	2.02	.46	5.36
Des Moines, Iowa	.18	2.08	1.38	.06	.94	2.52	.30	9.38
Keokuk, Iowa	.52		.06		T.	T.	.38	3.28
Sioux City, Iowa	.01	.47			2.27	.42	.08	6.21
Kansas City, Mo	.22	T.	.76	.01	.23	.68	1.0	5.10
St. Louis, Mo	.01	T.	.12		.40	T.	.42	1.61
Springfield, Mo		T.	T.	.20	.80	1.54	1.66	6.43
Hannibal, Mo	.06	T.	.04		.28	.32	1.61	3.96
Fort Smith, Ark		T.	.22	.02	1.96	.12	.44	4.75
Little Rock, Ark			T.	1.14	.68	.91	T.	3.27

In the northeast-central part of Kansas in the vicinity of Concordia, where the precipitation was greatest, it rained daily for fifteen days, a total of about 10 inches in depth.

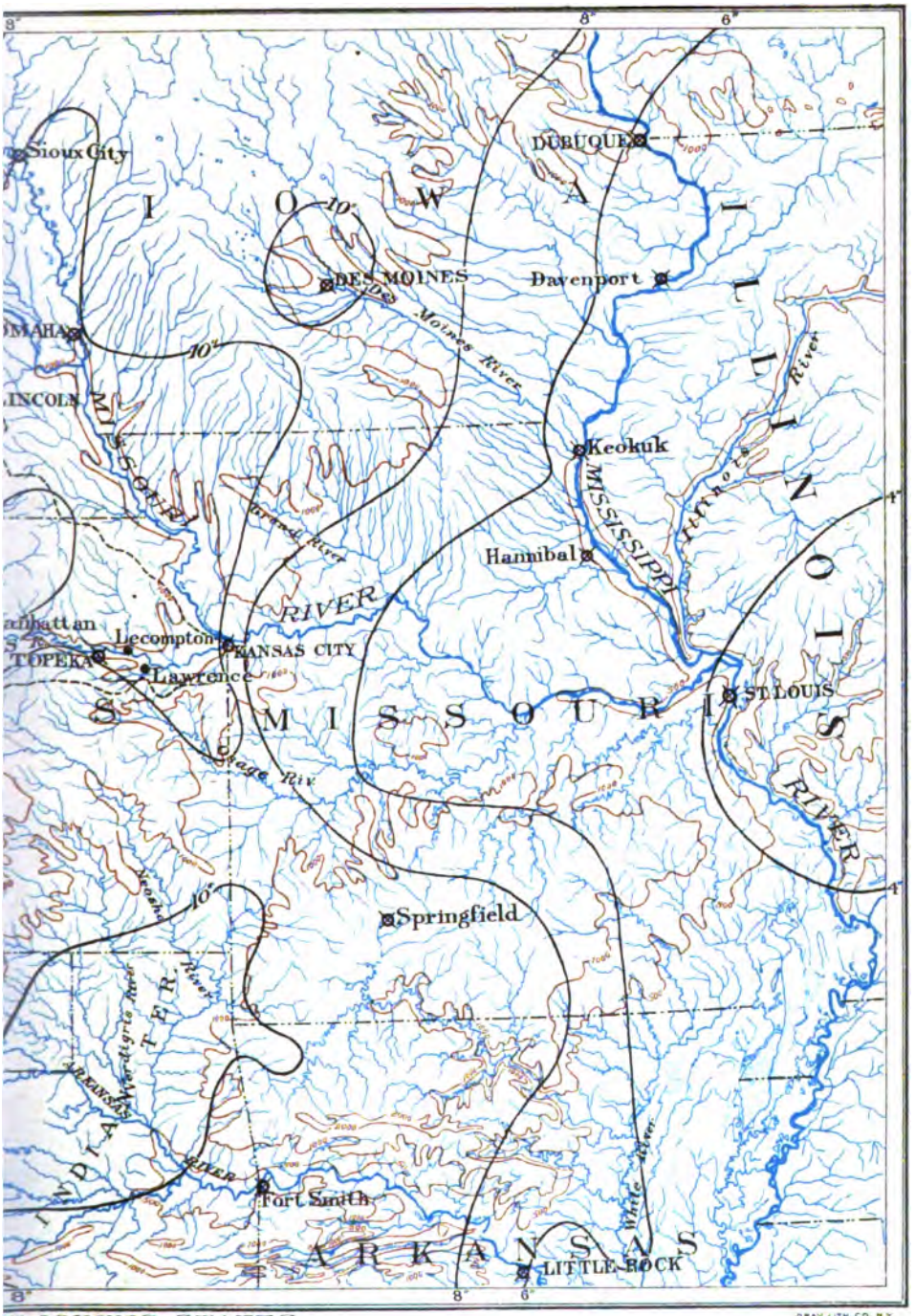
Rainfall during storm and during May compared.—The rainfall for the first half of May was considered above the normal for this month; the streams were carrying more than their normal amount, as can be seen from the gage readings in the table on pages 35, 36, and the ground storage available was small. It was this lack of surface and



MAP OF KANSAS
SHOWING THE WATERSHEDS OF THE KANSAS RIVER
ALSO LINES OF EQUAL RAINFALL FOR MAY, AND

100 50 0

Contour interval



DJOINING STATES

R STREAMS AFFECTED BY THE KANSAS FLOOD OF 1903

ATION OF U. S. GEOLOGICAL SURVEY GAGING STATIONS

100 200 miles

mean sea level

GRAV. LITH. CO. N.Y.

ground storage at the beginning of the flood period that made the flood of such exceptional magnitude.

The rainfall for May was over 17 inches at some stations. It was 17.34 inches at Salina, Kans.; 17.22 inches at Bradshaw, Nebr.; 15.45 inches at Thurman, Iowa; 17.29 inches at Blackburn, Okla. The area having a depth of 8 to 17 inches of rainfall during May covers the eastern two-thirds of Kansas, northern Indian Territory and Oklahoma, western Arkansas and Missouri, eastern two-thirds of Nebraska, western half of Iowa, and small parts of southern Wisconsin and southeastern Minnesota. The storm was more nearly central over Kansas River and tributary watersheds than over any other watershed of this area, hence the Kansas River flood was greater than that of any other stream. On the accompanying map (Pl. IV) are shown the watersheds affected, the lines of equal rainfall for May, and the location of the United States Geological Survey gaging stations.

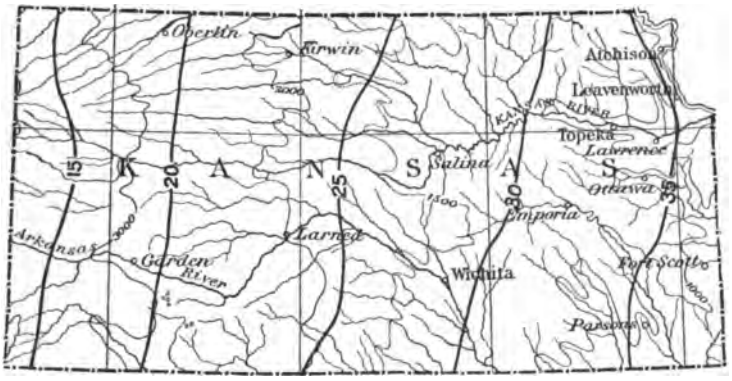


FIG. 2.—Mean annual rainfall lines over Kansas.

Curves of annual rainfall and rainfall during May compared.—Fig. 2 shows the lines of mean annual rainfall over Kansas. This mean annual rainfall is seen to decrease from about 35 inches in the eastern part of the Kansas River watershed to about 12 inches in the western part. More than half, therefore, of the ordinary mean annual rainfall over the eastern half of the Kansas River watershed fell during May, 1903, whereas, ordinarily, only about 14 per cent of this falls during that month.

CHARACTER OF WATERSHED FLOODED.

Location, extent, and topography.—Kansas River and its tributaries (Blue, Republican, Solomon, Saline, and Smoky Hill rivers), along which the greatest amount of damage was done by this flood, drain a long, comparatively narrow strip of country lying between Platte River on the north and Arkansas River on the south, and extending from Missouri River on the east to the foothills of the

Rocky Mountains in Colorado on the west. The length from east to west is 485 miles, and the width varies from 140 to 190 miles; the total area drained is 61,440 square miles—9,459 square miles in Colorado, 17,455 square miles in Nebraska, and 34,526 square miles in Kansas. This country is rolling prairie, with little timber except narrow strips along the streams in the eastern half of the State; the soil is very productive, especially in the river bottoms, and is mostly under cultivation where the rainfall is sufficient to mature crops. The eastern third of this area is covered with a tough buffalo-grass sod. The altitude on Gannett's contour map of the United States varies from 750 feet at Kansas City to 5,000 feet in the western part in Colorado. The 500-foot contour lines are shown in Pl. IV. The surface is seen to rise at a gradually increasing rate from east to west; the valleys are wide and comparatively shallow, with very little opportunity for storage of water.

Description of Kansas River.—Kansas River differs greatly from both the Platte on the north and the Arkansas on the south, in that it is wholly within the plains area and has no mountain drainage; its entire water supply comes from rain falling on the plains within its drainage area. The surface storage is very small, there being no lakes, ponds, or swamps to hold back the flow, and no mountains to hold back the winter precipitation. The minimum flow is, therefore, very small and the flood flow very large. The term "Kansas" is applied to this stream only after the Smoky Hill joins the Republican near Manhattan; the Solomon and the Saline are tributaries of the Smoky Hill. The principal tributaries of the Kansas are the Blue and the Delaware on the north and the Wakarusa on the south. The areas drained by these streams in the different States are given in the table below:

Drainage areas in Kansas River basin.

Basin.	Total area.	Area in Kansas.	Area in Nebraska.	Area in Colorado.
	Sq. miles.	*Sq. miles.*	*Sq. miles.*	*Sq. miles.*
Kansas River entire	61,440	34,526	17,455	9,459
Kansas River above Lawrence	59,841	33,927	17,455	9,459
Kansas River above Lecompton	58,550	31,636	17,455	9,459
Blue River above Manhattan	9,490	2,450	7,040	.....
Republican River entire	25,837	7,496	10,415	7,926
Republican River above Superior	22,347	.....	.....	.....
Smoky Hill River entire	20,428	18,895	.....	1,533
Smoky Hill River above Ellsworth	7,980	6,447	.....	.....
Saline River entire	3,311	3,311	.....	1,533
Solomon River entire	6,882	6,882	.....	.....
Solomon River above gaging station	6,815	.....	.....	.....
Kansas River (proper) above Lecompton	5,123	.....	.....	.....

Kansas River above Lawrence meanders through the bottom land, crossing and recrossing it, but as a rule being closer to the south border than to the north. This bottom land, which includes the land adjacent to the river and not more than 20 feet in elevation above it, is mainly alluvial matter and varies in width from 1 to 4 miles. The bed is mainly shifting, sandy, alluvial material, though in a few places it is rocky. The banks as a rule are not more than 10 feet in height except where the river runs near the bluffs, where they are from 30 to 40 feet high. They are easily eroded by the current, and frequently a new channel is formed and a considerable amount of land washed away.

THE FLOOD.

Reasons for magnitude of flood.—The unprecedented rainfall of 5 to 10 inches in sixteen days, falling on a watershed with practically no surface storage, and at a time when the flow of the streams was considerably above their normal and the ground well saturated with water, was sure to produce an enormous flood. The Kansas daily newspapers of the 24th to 27th report heavy rain and wind storms, with loss of life and property.

Flood in vicinity of Topeka.—On the 27th the Topeka Capital reports "200 homeless in Salina, Kans., from flood due to rain the previous night;" also several tornadoes in Kansas and Iowa, in which a number of persons were killed. The mean reading of the United States Geological Survey gage near Salina, on Saline River, for the 27th, was 32.6 feet, only about 1 foot less than the maximum during this flood. On the 28th the same paper reports the streams in Missouri, Iowa, Nebraska, and Oklahoma out of their banks. The mean daily gage reading at Lecompton for this date is 18 feet, about 1 foot higher than at any time during the previous twenty-two years. On May 29 the same paper reported "the worst flood situation that has ever confronted Kansas for many years; bridges and tracks under water; many families driven from their homes; thousands of acres of corn ruined, and greater floods expected on account of continued heavy rain." The mean daily gage reading at Lecompton for this date was 21.1 feet, which is 3½ feet higher than the highest in the twenty-two years' record. On the 30th the same paper reports "practically all of North Topeka submerged; 4,000 North Topeka persons driven from their homes, and 1,500 houses partly submerged." The mean daily gage height at Lecompton for this date is 24 feet, which is 6.4 feet higher than the highest reading in the twenty-two years' record. On Sunday, May 31, the flood in Kansas River reached its maximum height. The mean reading of the Lecompton gage for this date was 28.75 feet and the maximum 29.50, which is about 12 feet higher than the highest gage reading at this place in twenty-two years. The Topeka Capital of this date states that over the highest part of North Topeka the river was flowing 5 feet deep. The water had reached the eaves of many houses supposed to be above high water.

Flood in the vicinity of Kansas City.—Missouri River at Kansas City reached danger line—21 feet gage height—on the 27th of May. On the morning of the 31st it reached 27½ feet, and on June 1 it reached its highest point—35 feet—which is 14 feet above danger line and only 2 feet below the highest point reached by the great flood of June 20, 1844.

The Kansas City Times of June 1 described the condition in Kansas City as follows:

Kansas City passed yesterday the most disastrous day in its history; and to-day an emergency confronts its people greater than they have ever known before; problems, too, that affect the welfare of all of the city are to be met, and they must be met promptly. All Saturday night and all yesterday the torrents swept down the Missouri and Kansas until the valleys of the Kansas and Missouri were a solid mass of raging water from bluff to bluff. On the Union Depot platform there were 6 feet in depth of water, and all over the west bottoms, the center of the heavy wholesale and manufacturing business of the city, there was an equal or greater depth. Several lives it is known have been lost, and many more it is feared have perished. Hundreds were rescued from impending death, but late last night people standing on the west bluff could hear the agonizing cries for help from men and women in houses out on the wide expanse of water, but what the loss of life has been can not be accurately known until the water recedes. Early yesterday frame and other unstable structures in Argentine and Armourdale, that had been partly submerged for several days, began to leave their foundations and float away, thus a great mass of wreckage collected; it swept down on the bridges that spanned the Kansas. First the Kansas City Belt Line bridge above the stock yards went out, later the bridges at the stock yards and the Union Pacific Railroad bridge. The mass of wreckage grew. Soon it was joined by the great oil tank that had floated down from the Armourdale oil plant, which surged against the double-deak Central Avenue bridge, which carried the electric railway across to Kansas City, and the bridge went down as if made of paper. Five more bridges between this point and the mouth of the Kansas were swept away, and worst of all the bridge which carried the flow line of the Kansas City waterworks went down, carrying with it the 36-inch pipe which furnishes 200,000 people with their water supply.

In the Kansas City Times of June 10 is the following statement of the Executive Relief Committee of Kansas City, Kans:

Two entire wards and parts of two others, comprising, by the official census of March, 1903, at least 23,000 out of the population of 60,000 in the whole city, are made homeless and destitute. Hundreds of their houses have been swept away, and those that remain have been under 10 to 20 feet of water, and left with a deposit of silt both inside and out from 1 to 3 feet in depth. In most instances persons were able to save only the clothing they wore. Their household goods and clothing have been swept away or destroyed by the water and mud.

Flood in vicinity of Lawrence.—Some idea of the magnitude of the flood and its effect in the vicinity of Lawrence, Kans., can be gathered from the following notes taken from the Lawrence Daily Journal:

At 3 a. m., May 29, the pumping plant which supplies the city with water was compelled to shut down on account of the very high water, and it could not resume pumping again until June 6. At 8 a. m. the electric-light plant was

compelled to shut down. At noon the water broke through the Union Pacific embankment, flooding North Lawrence. The Santa Fe trains could not pass Lake View, 4 miles west of Lawrence, on the main line.

At 9 a. m., May 30, Congressman Bowersock's flouring mill was swept off from its foundation, and in a few minutes not a vestige of it was left. The flume also was destroyed, but the dam was practically uninjured.

Six hundred feet of the Eudora highway bridge was swept away this morning. * * * The water is running over the Lecompton highway bridge, and the floor is being taken up to save the structure being swept away. * * * During the day 50 buildings were counted as they passed over the dam.

June 1: The scene in North Lawrence is one of destruction and desolation. There is probably not a building standing on its foundation west of Bridge street, save the few structures north of the Usher block, and there is hardly a habitation of any kind for man or beast in that part of the city. The part north of the Union Pacific depot has also been swept, and numerous houses there have been carried from their foundations down the streets. The Griffin ice houses and their contents, the transfer barns and their equipment, including several head of horses, the Union Pacific freight depot, and many of the larger frame stores were carried down by the stream, while the foundations of brick and stone structures softened, and the crumbling ruins are all that is left to tell the story of their existence.

There was very little furniture or personal property saved. No opportunity was afforded to get live stock on higher ground. * * * At 5.30 Sunday evening the north span of the Lawrence highway bridge was swept from its piers by the house of Mr. William Parsons striking it as it was being swept away by the raging torrent. This house had only been deserted by Mr. Parsons and his family on Friday, and contained all of its furniture. If it had not been for the forethought of the authorities in uncoupling this bridge span from the others in anticipation of such an accident, the damage to the bridge might have been much greater.

Flood in vicinity of St. Louis.—At St. Louis the highest stage of the river was 38 feet, or half a foot higher than the flood of 1858 and 3.4 feet below the flood stage of 1844.

The Journal of Agriculture for June contains the following:

The great flood of 1844, although it reached a higher point, did comparatively little damage, as the localities covered by it were almost uninhabited at that time. Kansas City, with its great railroad and packing industries, was not even dreamed of and was bare prairie land. East St. Louis, which is now a big city engulfed in the flood, had no existence, but was simply an entrance to St. Louis. * * * So the former floods, however high their waters, could inflict but small loss compared with that now deluging so many thousand miles of fertile country and cities teeming with large population. The money loss involved is simply enormous, but the most lamentable feature is the loss of life. * * * The damage inflicted by these periodic inundations seems to be irremediable. The ingenuity of man has not yet been able to suggest any plan by which the excessive rainfall which causes floods can be subjected to control. To anyone who saw the mighty Missouri at its highest flood near the junction with the Mississippi the bare idea of storage reservoirs and dams is absurd. Its volume is so vast that probably no valley in our country could contain it, even if it were at all possible to restrain it by dams and barriers. The Missouri River defies restraint, and all that can possibly be done to mitigate its ravages when on the rampage is to be prepared for it generally and by building railroads and dwellings in future higher than its recorded heights.

Daily rainfall at Weather Bureau stations, May 16–31, 1903.—Continued.

Station.	25.	26.	27.	28.	29.	30.	31.	Total.
Williston, N. Dak		0.08		0.60	0.04			3.87
Bismarck, N. Dak		.20					0.50	3.09
Pierre, S. Dak		.04			.46	T.		1.71
Rapid, S. Dak	0.02			T.	.12		.16	1.14
Huron, S. Dak	.08	.24	0.01					1.57
Yankton, S. Dak		.39	.39	.43	.40	0.10	.03	4.55
St. Paul, Minn	.62	1.04	.24					2.65
Moorhead, Minn								3.10
Valentine, Nebr	.04	.50			.32	.02		1.48
North Platte, Nebr	.08	.18		.01	.43	.06		.98
Lincoln, Nebr	1.37	1.03	.43	.15	1.94	1.13	.10	8.09
Omaha, Nebr	.95	.93	.56	T.	.52	.40	.10	5.21
Concordia, Kans	.16	.96		1.64	3.68	.25	.42	10.59
Topeka, Kans	.22	.21	.28	.24	1.71	.26	.91	5.41
Wichita, Kans		.22		.22	.82	.02	.26	6.17
Dodge, Kans		.30	.26	.28	.03	.04		1.89
Denver, Colo	.01	T.		.01	.03		.28	.34
Pueblo, Colo		.06		.22	T.			.28
Cheyenne, Wyo		T.		.12	T.			.39
Oklahoma, Okla		T.		.18	3.26	.02	T.	9.38
Fort Worth, Tex		T.		.68	.46	T.		1.15
Dubuque, Iowa	.10	1.40	1.11	.02	T.	.34	.12	3.76
Davenport, Iowa	.08	.87	.42	.38	.32	2.02	.46	5.36
Des Moines, Iowa	.18	2.08	1.38	.06	.94	2.52	.30	9.38
Keokuk, Iowa	.52		.06		T.	T.	.38	3.28
Sioux City, Iowa	.01	.47			2.27	.42	.08	6.21
Kansas City, Mo	.22	T.	.76	.01	.23	.68	1.01	5.10
St. Louis, Mo	.01	T.	.12		.40	T.	.42	1.61
Springfield, Mo		T.	T.	.20	.80	1.54	1.66	6.43
Hannibal, Mo	.06	T.	.04		.28	.32	1.61	3.96
Fort Smith, Ark		T.	.22	.02	1.96	.12	.44	4.15
Little Rock, Ark			T.	1.14	.68	.91	T.	3.27

In the northeast-central part of Kansas in the vicinity of Concordia, where the precipitation was greatest, it rained daily for fifteen days, a total of about 10 inches in depth.

Rainfall during storm and during May compared.—The rainfall for the first half of May was considered above the normal for this month; the streams were carrying more than their normal amount, as can be seen from the gage readings in the table on pages 35, 36, and the ground storage available was small. It was this lack of surface and

Pacific bridge, which withstood the flood, is a very heavy, double-track bridge, and was loaded from end to end with locomotives. Some idea can be formed of the débris which came down the river from the amount which was lodged against the piers and abutments of this bridge. (See Pl. V, *B*.) The damaged bridges were nearly all steel structures of about 200 feet span, resting on masonry or metal piers. Several of them were double-track structures. The cost of replacing them is estimated to be from one and a quarter to one and a half million dollars. One of the bridges destroyed carried a 36-inch water-supply pipe for supplying Kansas City, Mo., and a part of Kansas City, Kans., with water. As a result of its failure, the whole city was without water for twelve days, and the electric-light company, gas company, and street railways were compelled to shut down for lack of water.

The highway bridge at Eudora was partly washed away. The highway bridge at Lawrence had one span washed away. The river at this point made a new channel around the north end of the bridge, washing away the Union Pacific freight station and several other structures. In September the total flow of the river was passing through this new channel.

The highway bridge at Lecompton was uninjured, but a channel was scoured around the north end of it. All the bridges across the Kansas at Topeka were destroyed except the Melan arch bridge, and the approaches of that were so badly damaged that for several days the only means of access between the north and south parts of the city was by boat.

By scouring action.—The damage due to the scouring action of the flood was very great, the railroad companies being the heaviest losers in this respect. Many miles of railway track located near Kansas River were washed away or badly undermined.

It can be shown mathematically that the increase in the scouring action of water and its transporting power is proportional to the sixth power of the velocity. If, for example, the velocity along the bed of a stream at a given point is 2 feet per second at ordinary stage, and during the flood the velocity is found to be 4 feet per second, then the cutting or scouring action of the water for the 4 feet velocity during flood is sixty-four times that of the 2 feet velocity at ordinary stage, and bowlders will be carried along during the flood that are sixty-four times as heavy as those carried at ordinary stage.

However, the power of a stream in flood to cut river banks and make new channels, as compared with its action at ordinary flow, varies at a much larger rate than as the sixth power of its velocity, because the toughness of a bank of alluvial material and its ability to resist scour decreases very rapidly from the bottom toward the top. Take, for example, the case of a stream flowing in a channel with banks 10 feet high. When the water is 2 feet deep it comes in contact

with only the lower 2 feet of the bank, which is tough and firm. When the river rises 3 feet, the water flows more rapidly and comes in contact with a part of the bank which is 5 feet up from the bed, and which is less tough and more readily scoured. When water rises to 8 feet or more, the velocity is several times greater than at the shallow depth and the water comes in contact with a part of the bank much more easily eroded than the portion near the bottom; hence the wear is very rapid during a flood, and in a few hours deep channels are cut and acres of land washed away. This cutting action is greatest just below a bend in the stream. Here water tends to continue moving in a straight course, and hence strikes the concave surface of the bank just below the bend. When, as during the recent flood, the river rises high above its banks, it takes a straight course, regardless of its banks, and often does much damage where it was supposed the water could never reach. The new channel through North Lawrence around the dam is 200 feet wide and so deep that the whole flow of the river passes through it. The bed of the Kansas at Lecompton Bridge was scoured to a depth of from 3 to 6 feet during the flood; a depth of several feet was scoured also for several hundred feet in width north of the north abutment. A large amount of material was scoured from the left bank of Republican River at the gaging station at Junction, and, in fact, in many places along the stream there is evidence of a very considerable amount of material having been carried away. In many places material has been deposited as well. Some of this deposit is fine silt which will enrich the land, but often it is sand which will injure it to a considerable extent.

HYDROGRAPHIC DATA OF KANSAS RIVERS.

METHOD OF OBTAINING DATA.

The United States Geological Survey has at present eight gaging stations in the Kansas River basin. Four of these are in the Republican River basin, two at Benkelman, Nebr., one at Superior, Nebr., near the State line, and one at the mouth of the river near Junction, Kans. There is one on the Smoky Hill River at Ellsworth, one on the Saline near its mouth, one on the Solomon near its mouth, one on the Blue near its mouth, and one on the Kansas at Lecompton. Six of these stations have been in operation about eight years. At each of these stations there is a gage for measuring the rise and fall of the water surface. This gage is read twice a day by a person living near by, and a copy of the reading is sent at the end of each week to a resident hydrographer who has charge of all the stations in that State. The resident hydrographer visits each station several times a year to measure the volume of water flowing and inspect the station. He measures the depth at 5-foot intervals across the stream and computes the area of the prism of water flowing.

He also measures the velocity at a sufficient number of points in the cross section to enable him to compute the mean velocity of the prism of flowing water. From the cross section and mean velocity he computes the volume passing down the stream for that particular gage height and stream stage. After he has obtained the volume of flow for all stages, he prepares a station rating table which gives the volume of flow for each gage reading. The mean volume flowing per day is then taken from this table, using as a gage reading the mean of the gage readings of the morning and evening. Having the mean daily flow, the monthly and yearly flow are easily computed. The tables (pp. 37–68) give the maximum and minimum monthly flow, as well as the mean, for purposes of comparison; they give the total flow in acre-feet, for purposes of storage; the run-off per square mile, to show the number of square miles necessary to yield a given flow per second, also the depth in inches of run-off, for the purpose of showing the proportion of rainfall that is flowing in the stream.

The table on pages 35, 36 gives the mean daily gage heights and corresponding estimated discharges for May and June, 1903, at seven of the gaging stations in the Kansas River basin. These flood discharges are very difficult to measure, or even to estimate. In fact, it was impossible to measure the velocity, or even to visit some of the stations during this flood. During floods there is always more or less backwater, and consequent retarding of velocity, so that usually the volume of water flowing is much less than the flooded land indicates; much of the overflow water has very little velocity, so that flood estimates made by eye are, as a rule, too large. In computing the discharges in the table the station-rating table has been extended to the greatest gage height, assuming that the banks were not overflowed, and a certain percentage of the discharge has been added, depending on the magnitude of the overflow and the velocity as shown by the scour. The difficulty of computing accurately the discharge during this flood is further complicated by the change in the bed of the stream during the flood. The station-rating curve for 1902 can not be used for computing the discharge for 1903, and the discharge measurements made in 1903 are too few to enable a good station-rating table to be prepared. At the Lecompton and Junction stations the river bed was scoured, so that the discharge after the flood was greater than before it for a given gage reading. At Salina, Niles, and Ellsworth the channel was filled in to such an extent at some point below the station that the discharge is less for a given gage reading since the flood than before it. The bed of the stream at the Manhattan station changed very little during this flood. The results given in the table are believed to be under- rather than over-estimates.

This table is arranged so as to show the increase in flow from the western part of the drainage basin to the eastern part. The run-off per square mile in May increases from 0.30 second-foot at Ellsworth

in the western part to 2.01 second-feet at Manhattan, and in June increases from 0.12 second-foot at Ellsworth to 1.73 second-feet at Manhattan.

Among the many interesting facts shown by the table the following may be noted: (1) The great volume of flood flow as compared with the ordinary flow. (2) The sudden changes in stream stage—as, for example, the change from 3.30 feet gage height on May 27, at Ellsworth to 15.40 feet on the 28th. (3) The large range of stage of stream—as, for example, the change from 5.90 to 36.50 feet at the station on Blue River.

It is seen from the Ellsworth record that this flood extended only a short distance west of Ellsworth, Kans., that is, it did not extend beyond the eastern two-thirds of the State of Kansas.

A TWELFTH STREET BRIDGE, ARGENTINE, KANS., AFTER THE FLOOD.

B. KANSAS CITY-SOUTHERN RAILROAD BRIDGE, KANSAS CITY, AFTER THE FLOOD.

Daily mean gage height and discharge in second-feet.

Day.	Smoky Hill River at Ellsworth, Kans.				Saline River at Salina, Kans.				Solomon River at Niles, Kans.				Republican River at Junction, Kans.			
	May.		June.		May.		June.		May.		June.		May.		June.	
	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
1	3.50	823	7.70	4,005	11.50	955	(a33.50)	7,895	15.40	2,429	32.90	10,208	7.17	6,450	17.30	44,280
2	3.00	557	7.00	3,410	15.90	1,712	(33.30)	7,805	15.60	2,491	33.30	10,383	5.90	3,400	16.22	40,380
3	3.00	557	5.40	2,129	13.90	1,353	(33.00)	7,670	12.80	1,645	33.80	10,602	4.87	1,780	13.85	31,860
4	2.90	510	4.80	1,690	13.30	1,251	32.70	7,535	10.50	997	33.60	10,514	4.32	1,155	11.32	22,680
5	5.00	1,835	4.40	1,410	11.20	910	32.00	7,220	10.20	915	32.80	10,165	4.12	945	10.60	20,160
6	5.30	2,054	4.30	1,342	13.00	1,200	31.20	6,868	9.90	833	32.00	9,815	3.95	787	9.35	13,050
7	6.30	2,829	3.90	1,075	11.30	925	30.60	6,590	10.40	970	30.00	8,941	3.92	735	8.22	9,600
8	5.90	2,512	3.60	883	13.30	1,251	28.50	5,645	10.70	1,052	25.60	7,018	3.90	735	7.35	7,050
9	5.50	2,204	3.50	823	15.80	1,694	24.70	3,935	16.80	2,877	20.80	4,296	3.90	735	6.80	5,400
10	5.00	1,835	3.40	765	16.70	1,856	20.20	2,488	20.00	4,000	18.80	3,568	3.85	682	6.47	4,525
11	4.90	1,762	3.30	710	16.50	1,820	17.80	1,645	21.80	4,590	18.10	3,320	4.40	1,260	6.20	4,000
12	4.40	1,410	3.20	657	15.40	1,622	17.20	1,555	23.10	5,153	18.50	3,460	8.42	10,200	5.97	3,500
13	5.50	2,204	3.20	657	13.90	1,353	16.50	1,450	23.20	5,191	18.00	3,285	10.85	21,060	5.72	3,020
14	5.60	2,280	3.10	606	16.10	1,748	15.80	1,345	25.10	6,800	16.60	2,807	12.27	26,100	5.45	2,600
15	4.50	1,479	3.00	557	17.30	1,964	15.30	1,270	28.20	8,155	15.90	2,588	10.05	18,180	5.40	2,520
16	4.00	1,141	2.90	510	17.50	2,000	14.90	1,210	29.50	8,723	15.50	2,480	8.55	10,650	5.25	2,287
17	3.80	1,010	2.80	465	18.00	2,090	14.50	1,150	32.10	9,859	15.10	2,396	7.62	7,800	5.15	2,148
18	3.60	888	2.70	422	17.20	1,948	14.10	1,090	31.00	9,378	14.70	2,215	7.20	6,600	5.02	1,980
19	3.40	765	2.60	381	18.60	1,302	13.80	1,045	25.00	6,756	14.40	2,124	7.87	8,550	4.92	1,840
20	3.20	657	2.80	465	12.30	1,061	13.50	1,000	18.20	3,355	14.20	2,065	6.52	4,650	4.75	1,660
21	3.00	557	2.80	465	13.30	1,251	14.00	1,075	17.50	3,115	14.00	2,005	6.37	4,300	4.75	1,660
22	3.10	606	2.60	381	20.10	2,469	14.10	1,090	21.50	4,557	13.70	1,915	6.72	5,150	4.67	1,540
23	3.50	823	2.80	465	23.10	3,215	13.50	1,000	20.20	4,074	13.40	1,825	8.90	11,700	4.60	1,480
24	3.20	657	2.80	465	20.20	2,488	13.60	1,015	17.70	3,183	13.20	1,765	8.17	9,450	4.52	1,370
25	3.00	557	2.70	422	18.00	2,090	13.50	1,000	18.00	3,285	13.50	1,855	7.35	7,050	4.60	1,480
26	5.00	1,835	3.40	765	28.70	5,735	13.20	955	20.30	4,111	13.80	1,945	9.20	12,600	5.48	2,680
27	3.30	710	3.90	1,075	32.60	7,490	12.70	880	25.00	6,756	13.80	1,945	11.50	23,400	6.07	3,700
28	15.40	10,866	3.00	557	30.70	6,635	12.40	835	23.30	5,229	15.70	2,522	13.10	29,160	5.47	2,600
29	16.00	11,392	2.80	465	(b31.40)	6,950	12.70	880	32.30	9,946	15.60	2,491	18.00	46,800	5.20	2,210
30	13.80	9,466	2.50	342	(23.10)	7,295	13.80	1,045	31.30	9,509	17.20	3,013	18.20	47,520	5.50	2,680
31	10.20	6,130	.....	.....	(32.80)	7,580	.....	.....	32.20	9,902	.....	.....	17.75	45,900	.....	.....
Total	.....	73,021	.....	28,109	.....	83,255	.....	86,186	.....	149,880	.....	133,414	.....	375,484	.....	244,860
Mean	.....	2,356	.....	937	.....	2,685	.....	2,873	.....	4,835	.....	4,447	.....	12,112	.....	8,162
Run-off per square mile.	.....	0.30	.....	0.12	.....	0.81	.....	0.87	.....	0.71	.....	0.65	.....	0.47	.....	0.31

a Maximum gage height, 33.90 feet.

b Gage heights marked thus ( ) are interpolated.

Daily mean gage height and discharge in second-feet—Continued.

Day.	Republican River at Superior, Nebr.				Blue River at Manhattan, Kans.				Kansas River at Lecompton, Kans.			
	May.		June.		May.		June.		May.		June.	
	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
1	0.80	840	2.60	6,355	11.00	5,860	(35.00)	66,170	4.00	5,000	27.20	205,500
2	.86	940	2.10	4,240	11.50	6,490	(33.80)	63,050	4.00	5,000	26.25	198,500
3	.96	1,060	1.89	3,510	9.50	4,145	32.50	59,670	6.90	14,170	26.35	197,500
4	.80	840	1.76	3,015	7.20	2,060	30.00	53,170	6.70	13,490	24.35	177,500
5	.95	1,100	1.58	2,590	6.80	1,770	27.00	45,370	6.40	12,470	23.05	164,500
6	.99	1,150	1.45	2,290	6.60	1,630	25.50	41,470	6.40	12,470	20.50	138,500
7	1.09	1,470	1.33	2,055	6.60	1,630	21.50	28,900	6.30	12,130	18.20	115,500
8	2.02	3,805	1.44	2,310	6.40	1,490	17.50	15,900	6.15	11,620	16.50	98,500
9	2.08	4,480	1.41	2,205	6.20	1,350	14.60	10,880	5.90	10,770	14.80	81,500
10	2.59	6,480	1.20	1,610	6.00	1,220	13.30	8,980	5.95	10,940	13.85	72,500
11	3.20	9,415	1.19	1,595	5.90	1,155	12.40	7,700	6.30	12,130	12.70	60,500
12	2.75	7,995	1.21	1,640	6.30	1,420	11.90	7,015	7.50	16,250	11.55	49,500
13	2.60	6,900	1.13	1,560	11.90	7,015	11.30	6,230	9.25	22,375	10.45	38,900
14	2.32	4,950	1.08	1,350	18.30	17,500	10.70	5,500	10.80	29,000	9.85	33,725
15	2.08	4,155	1.07	1,295	18.60	18,100	10.40	5,150	13.10	42,475	9.30	29,000
16	1.99	3,795	1.04	1,260	12.80	7,560	10.10	4,800	12.45	38,312	8.90	26,100
17	1.68	2,910	1.00	1,070	10.40	5,150	9.80	4,470	10.25	26,250	8.50	23,400
18	1.42	2,325	.99	1,060	9.70	4,360	9.50	4,145	9.65	23,775	8.05	20,900
19	1.41	2,155	.96	1,005	9.00	3,630	9.30	3,935	9.40	22,900	7.80	19,150
20	1.33	2,040	.91	920	8.60	3,240	11.10	5,985	9.80	24,300	7.45	17,525
21	1.20	1,690	.96	975	11.70	6,750	10.60	5,380	9.60	23,600	7.00	15,050
22	1.54	2,545	.94	880	22.00	24,900	10.00	4,690	9.95	24,825	6.65	13,725
23	1.41	2,165	1.09	1,080	23.00	26,900	9.50	4,145	12.90	41,125	6.40	12,500
24	1.30	1,990	1.08	1,070	25.00	40,170	9.10	3,730	14.40	52,600	6.00	11,000
25	3.48	11,220	1.16	1,170	27.50	46,670	9.30	3,935	14.15	80,075	6.20	11,750
26	2.80	7,205	1.29	1,465	25.00	40,170	12.80	8,260	12.15	55,500	7.80	19,150
27	2.32	5,060	1.76	2,545	30.50	54,470	12.20	7,420	16.60	99,500	9.30	29,000
28	1.96	3,665	1.58	2,165	32.00	58,370	9.80	4,470	18.00	113,500	9.30	29,000
29	3.88	13,275	1.41	1,990	(33.50)	62,270	9.30	3,935	21.10	144,500	8.90	26,100
30	3.76	13,090	1.19	1,260	(34.80)	65,650	8.80	3,430	24.00	173,500	8.55	24,050
31	2.88	7,850	.....	.....	[a](36.00)	68,770	.....	.....	[b]28.75	221,000	.....	.....
Total	.....	137,810	.....	57,175	.....	591,865	.....	492,815	.....	1,396,552	.....	1,958,025
Mean	.....	4,445	.....	1,906	.....	19,092	.....	16,427	.....	45,018	.....	65,268
Run-off per square mile	.....	0.20	.....	0.09	.....	2.01	.....	1.73	.....	0.77	.....	1.11

[a] Maximum gage height, 36.50 feet.

[b] Maximum gage height, 29.50 feet.

A. BRIDGE AT LAWRENCE, KANS.

B. BRIDGE AND GAGE, LECOMPTON, KANS.

DESCRIPTION OF STATIONS AND DISCHARGE TABLES.

KANSAS RIVER AT LAWRENCE AND LECOMPTON, KANS.

The first gaging station on Kansas River was established in 1880 by Mr. J. D. Bowersock at the highway bridge at Lawrence, Kans. The gage consisted of a vertical timber marked to feet and inches and fastened to the lower side of the pier adjacent to the right bank. This gage was read daily, or oftener, and the readings recorded whenever there was any material change in them until August, 1895, when this place was selected as a United States Geological Survey gaging station. From this date to April 16, 1899, gage readings were taken twice daily, a station-rating curve prepared, and the run-off computed. On account of the sluggish flow at this place during low stages it was deemed advisable to move this station to the highway bridge at Lecompton. This bridge and the gage are shown in Pl. VII, *B*, p. 34. The channel at this place is about 800 feet wide, broken by four metal piers. The bridge is only a short distance from a bend and is somewhat oblique to the course of the stream. The bed is mainly sand and was scoured out a considerable amount during the recent flood. The gage is a 2 by 4 inch timber bolted to the metal pier adjacent to the right bank. The pier on the right bank is marked to half feet for high-water gage readings. The low-water flow is practically the same at Lecompton as at Lawrence, but during high stages Delaware River contributes a considerable amount between these places.

The following tables give run-off at Lawrence and Lecompton:

Estimated monthly discharge of Kansas River at Lawrence, Kans.

[Drainage area 59,841 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1891.[a]						
January	2,123	967	1,445	88,850	0.02	0.02
February	7,400	1,098	3,538	196,491	.06	.06
March	11,440	1,437	6,144	377,780	.10	.12
April	12,550	5,207	8,455	503,107	.14	.16
May	32,290	4,037	10,560	649,309	.18	.21
June	35,750	13,120	23,102	1,374,664	.39	.44
July	19,370	9,440	13,846	851,357	.23	.26
August	9,440	3,673	5,444	334,739	.09	.10
September	6,490	1,255	2,768	164,707	.05	.60
October	10,900	2,687	5,925	364,314	.10	.12

[a] Bull. U. S. Geol. Survey No. 140, 1896, pp. 151-152.

Estimated monthly discharge of Kansas River at Lawrence, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1891.						
November	2,998	1,872	2,382	141,739	0.04	0.04
December	4,803	1,870	3,129	192,395	.05	.06
The year	35,750	967	7,228	5,239,452	.12	2.19
1892.[a]						
January	2,395	1,255	1,669	102,623	.03	.03
February	8,350	2,395	5,292	304,399	.09	.09
March	23,600	6,940	12,581	773,576	.21	.24
April	17,390	7,870	12,712	756,417	.21	.23
May		14,890	37,005	2,275,348	.61	.70
June	14,890	2,687	8,134	484,007	.10	.11
July	2,687	1,437	2,059	126,603	.03	.03
August	3,673	1,255	2,010	123,590	.03	.03
September	2,998	787	1,448	86,262	.02	.02
October	1,643	787	1,042	64,070	.02	.02
November	1,872	1,098	1,487	88,483	.02	.02
December	1,643	787	1,225	75,332	.02	.02
The year		787	7,222	5,260,700	.12	1.54
1893.[a]						
January	1,437	1,098	1,278	78,581	.02	.02
February	6,490	1,255	3,650	202,711	.06	.06
March	2,123	1,098	1,339	82,332	.02	.02
April	2,998	1,255	2,011	119,663	.03	.03
May	5,207	1,643	3,209	197,314	.05	.06
June	21,440	3,327	9,284	553,031	.16	.18
July	21,440	3,327	8,474	521,046	.14	.16
August	15,500	1,643	5,980	368,311	.10	.12
September	6,940	787	2,072	123,293	.03	.03
October	10,900	1,098	3,663	225,344	.06	.07
November	1,872	1,098	1,426	84,853	.02	.02
December	1,872	787	1,021	62,779	.02	.02
The year	21,440	787	3,617	2,619,758	.06	.79
1894.[a]						
January	1,643	787	1,230	75,630	.02	.02
February	2,123	863	1,315	78,248	.02	.02

[a] Bull. U. S. Geol. Survey No. 140, 1896, pp. 151-153.

Estimated monthly discharge of Kansas River at Lawrence, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1894.						
March	4,035	1,255	2,134	131,215	0.04	0.04
April	7,400	1,437	2,903	172,741	.06	.05
May	2,123	787	1,208	74,277	.02	.02
June	28,990	1,255	9,044	538,155	.17	.15
July	11,990	1,437	4,740	291,451	.09	.08
August	2,687	787	1,478	90,879	.03	.03
September	2,123	967	1,445	85,984	.02	.02
October	3,327	787	1,189	73,109	.02	.02
November[a]			787	46,830	.01	.01
December[a]			550	33,818	.01	.01
The year			2,326	1,692,337	.51	.04
1895.[b]						
January[a]			500	33,818	.01	.01
February[a]	1,098	550	680	37,765	.01	.01
March	1,643	967	1,364	83,869	.02	.02
April	1,872	967	1,372	81,640	.02	.02
May	1,437	787	1,074	66,038	.02	.02
June	20,050	967	5,927	352,681	.10	.11
July	10,370	2,395	5,666	348,389	.10	.12
August	28,190	3,163	11,318	695,917	.19	.22
September	20,050	1,255	5,088	302,757	.09	.10
October	1,846	967	1,090	67,022	.02	.02
November	1,846	863	1,044	62,122	.02	.02
December	1,255	863	926	56,938	.02	.02
The year			3,008	2,188,956	.05	.69
1896.[c]						
January	1,540	698	973	59,827	.016	.018
February	1,540	1,255	1,400	80,529	.023	.025
March	1,437	825	1,230	75,630	.021	.024
April	12,550	1,176	4,447	264,615	.074	.082
May	34,158	4,223	13,944	857,389	.230	.260
June	38,583	5,546	14,809	882,980	.250	.280
July	53,308	5,546	23,408	1,439,311	.390	.450
August	22,688	3,956	8,935	549,395	.150	.170

[a] Approximate.
[b] Bull. U. S. Geol. Survey No. 140, 1896, pp. 151–152.
[c] Eighteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1897, p. 222.

Estimated monthly discharge of Kansas River at Lawrence, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1896.						
September	12,550	2,123	4,436	263,960	0.074	0.082
October	7,170	1,437	1,960	120,516	.033	.038
November	11,440	2,123	3,724	221,593	.062	.069
December	3,827	2,123	2,477	152,306	.041	.047
The year	53,308	825	6,814	4,968,051	.114	1.545
1897.[a]						
January	2,687	967	2,137	131,399	.035	.040
February	11,440	1,643	4,720	262,136	.079	.082
March	5,887	2,687	3,515	216,130	.059	.068
April	58,000	4,035	13,782	820,085	.230	.256
May	14,890	3,327	5,907	363,200	.099	.114
June	32,290	2,395	7,321	435,630	.122	.136
July	39,000	2,998	10,420	640,700	.174	.201
August	5,623	897	2,904	178,560	.049	.056
September	1,255	715	969	57,660	.016	.018
October	1,872	787	1,156	71,080	.019	.022
November	2,687	662	1,927	114,664	.032	.036
December	1,255	562	775	47,652	.013	.015
The year	58,000	562	4,628	3,338,896	.077	1.044
1898.[b]						
January	1,255	863	1,125	69,174	.02	.02
February	6,490	692	2,700	149,950	.05	.05
March	6,490	1,643	2,412	148,309	.04	.04
April	6,490	1,643	3,497	208,085	.06	.07
May	26,235	5,207	14,555	894,958	.24	.28
June	28,990	4,803	13,684	814,253	.23	.26
July	4,803	1,255	2,570	158,024	.04	.05
August	2,395	692	1,008	61,980	.02	.02
September	2,687	692	1,397	83,127	.02	.03
October	1,177	592	825	50,728	.01	.02
November	1,032	517	746	44,390	.01	.01
December	4,035	507	1,448	89,035	.02	.02
The year	28,990	507	3,831	2,772,013	.06	.87

[a] Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1899, p. 350.
[b] Twentieth Ann. Rept. U. S. Geol. Survey, pt. 4, 1900, p. 322.

A. VIEW OF THE KANSAS RIVER, DURING FLOOD. TAKEN FROM TOP OF THE NATIONAL BANK, LECOMPTON, KANS.

B "MAD WATERS," LAWRENCE, KANS., DURING FLOOD.

Estimated monthly discharge of Kansas River at Lecompton, Kans.

[Drainage area, 58,550 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1899.[a]						
April[b]	----	----	2,693	160,245	0.05	0.06
May	16,800	2,380	5,455	335,415	.09	.10
June	24,800	3,600	10,609	631,279	.18	.20
July	30,250	3,865	9,771	600,795	.17	.20
August	8,335	2,720	4,855	298,522	.08	.09
September	2,580	1,850	2,154	128,172	.04	.05
October	3,150	1,750	1,988	119,163	.03	.03
November	3,000	1,950	2,236	133,051	.04	.05
December	3,150	1,850	2,379	146,279	.04	.05
1900.[c]						
January	3,481	2,960	3,421	210,349	.06	.07
February	3,613	2,830	3,234	179,607	.06	.06
March	24,898	3,220	7,807	480,034	.13	.15
April	9,520	4,150	5,896	350,836	.10	.11
May	12,220	6,045	7,965	489,749	.14	.16
June	9,200	3,880	6,303	375,055	.11	.12
July	18,722	3,613	7,340	451,319	.13	.15
August	5,095	3,481	4,455	273,927	.08	.09
September	15,635	3,880	6,947	413,375	.12	.13
October	7,810	3,746	4,913	302,089	.08	.09
November	8,110	3,613	4,622	275,028	.08	.09
December	3,613	3,350	3,418	210,165	.06	.07
The year	24,898	2,830	5,527	4,011,533	.10	1.29
1901.[d]						
January	2,375	1,210	1,772	108,956	.03	.03
February	8,820	1,210	3,142	174,497	.05	.05
March	11,110	4,465	6,353	390,631	.11	.13
April	25,000	4,332	12,844	764,271	.22	.25
May	7,005	2,625	4,767	293,111	.08	.09
June	4,465	2,375	3,231	192,258	.06	.07
July	2,625	275	1,123	69,051	.02	.02
August	4,465	275	2,173	133,613	.04	.05

[a] Twenty-first Ann. Rept. U. S. Geol. Survey, pt. 4, 1901, p. 228.
[b] For entire month, based on gage heights of April 16 to 30.
[c] Twenty-second Ann. Rept. U. S. Geol. Survey, pt. 4, 1902, p. 338.
[d] Water-Sup. and Irr. Paper No. 75, U. S. Geol. Survey, 1903, p. 144.

Estimated monthly discharge of Kansas River at Lecompton, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1901.						
September	6,710	2,375	4,367	259,855	0.07	0.08
October	5,000	445	2,270	139,577	.04	.05
November	1,885	275	1,222	72,714	.02	.02
December	1,000	625	805	49,498	.01	.01
The year	25,000	275	3,672	2,648,032	.06	.85
1902.[a]						
January	1,000	445	767	47,161	.01	.01
February	625	275	492	27,324	.01	.01
March	3,675	100	2,460	151,259	.04	.05
April	3,415	2,250	2,653	157,864	.05	.06
May	23,075	2,125	8,435	518,648	.14	.16
June	28,500	11,110	17,131	1,019,365	.29	.32
July	81,400	11,450	38,291	2,354,422	.65	.75
August	40,812	6,415	11,941	734,223	.20	.23
September	53,050	5,270	14,990	891,977	.26	.29
October	42,112	9,130	17,734	1,090,421	.30	.35
November	13,320	9,450	10,830	644,430	.18	.20
December	15,900	9,130	11,785	724,631	.20	.23
The year	81,400	100	11,459	8,361,725	.20	2.66
1903.						
January	14,170	11,100	12,613	775,542	.22	.25
February	13,830	9,130	11,035	612,883	.19	.20
March	25,000	9,610	16,156	993,394	.28	.32
April	10,265	5,275	7,182	427,359	.12	.13
May	221,000	5,000	45,018	2,768,049	.77	.89
June	205,500	11,000	65,268	3,883,715	1.11	1.34
July	20,900	11,750	14,085	866,052	.24	.28
August	58,500	11,187	27,568	1,695,090	.47	.54
September	18,325	2,635	8,631	513,580	.15	.17
October	5,000	2,375	3,698	227,381	.063	.073
November	12,500	5,692	10,182	605,871	.17	.19
December	8,205	5,000	6,472	397,948	.11	.13
The year	221,000	2,375	18,992	13,766,834	.324	4.513

[a] Water-Sup. and Irr. Paper No. 84, U. S. Geol. Survey, 1903, p. 97.

The hydrographs of this river at this station (see Pl. XII, *A*, p. 52) show very clearly how the discharge varies throughout the year. The minimum flow occurs in December or January, during very cold weather, and has been less than 600 second-feet. The maximum flow occurs, as a rule, some time between May 1 and July 31, and hence is due to rain and not melting snow. The monthly run-off per square mile prior to May, 1903, varies from 0.01 to 0.60 second-foot and the yearly run-off per square mile from 0.04 to 0.20 second-foot. In June, 1903, during the flood, however, the run-off per square mile was 1.11 second-feet, and the run-off for the year was 0.324 second-foot. The maximum monthly run-off is 129 times the minimum monthly run-off.

BLUE RIVER NEAR MANHATTAN, KANS.

The station on Blue River is at a highway bridge 4 miles north of Manhattan. It was established April 12, 1895, and has been in operation since that date. The channel is straight for several hundred feet above and below the bridge and is 225 feet wide at ordinary stage, without piers. The bed is mainly clay and changes very little. The gage is a vertical timber, marked to feet and tenths of a foot. The left bank is subject to overflow. During the flood of May and June, 1903, the water was 3 feet deep on the bridge, and the drift and a considerable portion of the flood, estimated at 60 per cent of the flow, passed around the bridge.

The following table gives the run-off data obtained at this station:

Estimated monthly discharge of Blue River near Manhattan, Kans.

[Drainage area, 9,490 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1895.[a]						
April 14 to 30	605	502	548	18,479	0.06	0.04
May	986	431	552	33,941	.06	.07
June	7,500	552	1,604	95,444	.17	.19
July	2,301	527	756	46,485	.09	.10
August	9,900	478	2,036	125,190	.21	.24
September	8,550	552	1,207	71,822	.13	.15
October	552	385	495	30,436	.05	.06
November	605	385	531	31,597	.06	.07
December	605	502	577	35,478	.06	.07

[a] Eighteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1897, p. 218.

Estimated monthly discharge of Blue River near Manhattan, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1896.[a]						
January	605	515	568	34,925	0.06	0.07
February	619	502	567	32,614	.06	.06
March[b]						
April 16 to 30	5,283	1,115	2,196	65,340	.23	.13
May	13,725	1,328	4,800	295,142	.51	.59
June	19,425	677	4,594	273,362	.49	.55
July	21,975	888	5,494	337,815	.58	.67
August	12,375	1,007	2,659	163,497	.28	.31
September	4,724	868	1,621	96,456	.17	.19
October	7,725	363	1,179	72,494	.12	.13
November	4,724	527	1,189	70,751	.13	.14
December	1,028	408	786	48,329	.08	.09
1897.[c]						
January	1,300	610	965	59,335	.10	.12
February	2,200	610	1,228	68,200	.13	.14
March	2,165	870	1,434	88,174	.15	.17
April	32,256	1,200	7,935	472,166	.84	.93
May	4,815	1,375	2,232	137,240	.24	.28
June	20,694	810	2,966	176,490	.31	.35
July	8,158	1,010	2,555	157,100	.27	.31
August	3,392	890	1,698	104,405	.18	.21
September	970	210	578	34,395	.06	.07
October	770	320	561	34,494	.06	.07
November	810	505	681	40,523	.07	.08
December	850	610	730	44,886	.08	.09
The year	32,256	210	1,964	1,417,408	.21	2.82
1898.[d]						
January	830	555	732	45,009	.08	.08
February	5,319	500	2,078	115,406	.22	.23
March	1,050	527	765	47,028	.08	.09
April	3,335	610	1,485	88,363	.16	.18
May	8,548	1,522	3,491	214,653	.37	.43
June	10,774	1,081	4,496	267,530	.47	.53

[a] Eighteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1897, p. 218.
[b] No record.
[c] Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1899, p. 348.
[d] Twentieth Ann. Rept. U. S. Geol. Survey, pt. 4, 1900, p. 319.

A UNDERMINED SECTION OF RAILROAD NEAR LAWRENCE, KANS.

B. RAILROAD TRACK ON TOP OF COAL CAR NEAR LAWRENCE, KANS.

Estimated monthly discharge of Blue River near Manhattan, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1898.						
July	1,333	500	740	45,501	0.08	0.09
August	610	230	454	27,916	.05	.06
September	665	280	377	22,433	.04	.04
October	637	207	360	22,136	.04	.04
November	555	342	443	26,361	.05	.06
December	775	432	615	37,815	.06	.07
The year	10,774	207	1,336	960,151	.14	1.90
1899.[a]						
January	910	680	721	44,333	.08	.09
February	1,080	770	876	48,651	.09	.09
March	7,240	792	1,845	113,445	.19	.22
April	1,325	702	944	56,172	.10	.11
May	29,693	635	4,081	250,931	.43	.49
June	30,030	1,325	4,767	283,656	.50	.56
July	10,371	885	2,699	165,955	.28	.32
August	6,367	305	1,532	94,199	.16	.18
September	635	400	498	29,633	.05	.06
October	2,212	287	597	36,708	.06	.07
November	612	460	536	31,894	.06	.07
December	747	380	524	32,220	.06	.07
The year	30,030	287	1,635	1,187,797	.17	2.33
1900.[b]						
January	960	270	619	38,061	.07	.08
February	815	460	585	32,489	.06	.06
March	5,705	657	1,388	85,345	.15	.17
April	4,457	522	1,392	82,830	.15	.17
May	8,240	1,160	2,736	168,230	.29	.33
June	1,690	590	967	57,540	.10	.11
July	23,280	400	2,580	158,638	.27	.31
August	2,927	500	973	59,827	.10	.12
September	4,410	522	1,812	107,821	.19	.21
October	1,110	545	731	44,947	.08	.09

[a] Twenty-first Ann. Rept. U. S. Geol. Survey, pt. 4, 1901, p. 227.
[b] Twenty-second Ann. Rept. U. S. Geol. Survey, pt. 4, 1902, p. 337.

Estimated monthly discharge of Blue River near Manhattan, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1900.						
November	1,500	590	727	43,259	0.08	0.09
December	635	460	589	36,216	.06	.07
The year	23,280	270	1,258	915,203	.13	1.81
1901.[a]						
January	815	522	682	41,935	.07	.08
February	4,082	635	1,471	81,695	.16	.17
March	3,013	770	1,129	69,420	.12	.14
April	13,155	1,010	2,146	127,696	.23	.26
May	2,325	725	1,152	70,834	.12	.14
June	1,500	460	828	49,269	.09	.10
July	770	305	473	29,084	.05	.06
August	2,800	380	775	47,653	.08	.09
September	1,010	305	511	30,407	.05	.06
October	590	380	470	28,899	.05	.06
November	1,325	460	590	35,107	.06	.07
December	680	380	498	30,621	.05	.06
The year	13,155	305	894	642,620	.09	1.29
1902.[b]						
January	560	408	461	28,346	.05	.06
February	910	408	619	34,378	.07	.07
March	910	480	618	37,983	.07	.08
April	560	408	483	28,740	.05	.06
May	17,510	408	2,157	132,629	.23	.27
June	17,510	910	6,298	374,757	.66	.74
July	43,430	4,540	16,866	1,037,050	1.78	2.05
August	27,686	1,790	4,329	266,180	.46	.53
September	29,990	1,460	5,011	298,175	.53	.59
October	20,006	1,790	5,071	311,804	.53	.61
November	4,740	1,526	1,945	115,735	.20	.22
December	3,136	1,066	1,897	116,642	.20	.23
The year	43,430	408	3,813	2,782,419	.40	5.51

[a] Water-Sup. and Irr. Paper No. 75, U. S. Geol. Survey, 1903, p. 143.
[b] Water-Sup. and Irr. Paper No. 84, U. S. Geol. Survey, 1903, p. 99.

Estimated monthly discharge of Blue River near Manhattan, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1903.						
January	2,710	875	1,542	94,814	0.16	0.18
February	11,460	785	2,373	131,790	.25	.26
March	9,815	1,220	4,929	303,072	.52	.60
April	1,560	785	1,046	62,241	.11	.12
May	68,770	1,220	19,092	1,173,920	2.01	2.42
June	66,170	3,480	16,427	977,474	1.73	1.93
July	4,580	1,700	2,644	162,573	.28	.32
August	34,710	1,840	9,626	591,880	1.01	1.16
September	6,235	1,155	2,576	153,282	.27	.30
October	2,710	925	1,337	82,209	.14	.16
November	5,860	1,035	1,849	50,519	.19	.21
December	2,290	785	1,044	64,193	.11	.13
The year	68,770	785	5,374	3,847,967	.565	7.79

The hydrograph of this stream at the gaging station is shown on Pl. XII, *B*, p. 52. It resembles to some extent that for the station on Kansas River at Lawrence, the minimum discharge occurring in December or January, and the maximum some time between April 1 and August 31. The monthly run-off per square mile prior to 1903 varied from 0.04 to 1.78 second-feet, and the yearly run-off per square mile from 0.09 to 0.40 second-foot. In May, 1903, the run-off per square mile was 2.01 second-feet, and the run-off per square mile for the year 1903 was 0.565 second-foot. The maximum monthly discharge is 53 times the minimum monthly discharge.

This stream is seen to have a more steady flow than Kansas River. This is because the Blue has a larger rainfall over its whole drainage area than the Kansas. In the Blue River watershed the mean annual rainfall is about 30 inches, and in the Kansas River basin it varies from 35 inches in the eastern part to 12 inches in the western part.

REPUBLICAN RIVER AT JUNCTION, KANS.

There are four stations on Republican River. One is at Junction, Kans., one at Superior, and two at Benkelman, Nebr. The station at Junction, Kans., is at Washington street highway bridge. It was established April 26, 1895, and has been maintained continuously since that time. This bridge is at a large bend in the stream. At low stages the channel is straight for 200 feet above and below the

station. The width is about 375 feet, broken by three piers. The bed is sandy and somewhat changeable. The velocity is rapid. The gage is a vertical timber marked to feet and tenths of a foot. During the flood of May and June a portion of the flow, estimated at 50 per cent of the total, passed around the left abutment and scoured out a channel about 4 feet deep and 200 to 300 feet in width.

The run-off results obtained at this station are given in the following table:

Estimated monthly discharge of Republican River at Junction, Kans.

[Drainage area, 25,837 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1895.[a]						
July	6,000	439	1,800	110,678	0.070	0.081
August	[b]25,000	265	3,192	196,269	.124	.143
September	4,170	219	542	32,251	.021	.023
October	326	220	249	15,310	.010	.012
November	355	230	282	16,779	.011	.012
December	575	230	392	24,103	.015	.017
1896.[a]						
January	556	392	448	27,547	.017	.020
February	1,131	504	756	43,486	.029	.031
March	504	504	504	30,990	.019	.022
April	3,770	485	1,414	84,139	.055	.061
May	3,920	785	2,124	130,601	.082	.095
June	3,980	494	1,370	81,520	.053	.059
July	[b]37,500	594	3,678	226,153	.142	.164
August	2,340	459	893	54,909	.034	.039
September	2,425	326	667	39,690	.026	.029
October	2,180	356	475	29,207	.018	.021
November	1,480	459	655	38,976	.025	.028
December	1,121	594	810	49,805	.031	.036
The year	[b]37,500	326	1,150	837,023	.044	.601
1897.[c]						
January	1,160	560	740	45,507	.029	.035
February	3,055	625	1,343	74,563	.052	.054
March	1,160	750	898	55,228	.035	.040

[a] Eighteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1897, p. 205.
[b] Estimated.
[c] Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1899, p. 340.

A. BRIDGE STREET, NORTH LAWRENCE, KANS., DURING THE FLOOD.

B. PILE DRIVER CONSTRUCTING TEMPORARY BRIDGE OVER NEW NORTH CHANNEL, LAWRENCE, KANS.

Estimated monthly discharge of Republican River at Junction, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1897.						
April	8,250	1,000	2,032	120,895	0.078	0.087
May	2,550	325	833	51,213	.032	.037
June	6,053	290	1,592	94,713	.062	.068
July	6,360	440	1,824	112,147	.071	.082
August	2,210	254	619	38,042	.024	.028
September	537	125	204	12,156	.008	.009
October	415	85	205	12,605	.008	.009
November	1,120	290	632	37,577	.024	.027
December	930	290	668	41,070	.026	.030
The year	6,360	85	966	695,716	.037	.504
1898.[a]						
January	900	475	732	45,009	.03	.03
February	1,895	645	1,034	57,425	.04	.04
March	825	475	656	40,336	.03	.03
April	1,310	590	804	47,841	.03	.03
May	7,175	862	2,126	130,723	.08	.09
June	6,165	700	2,125	126,446	.08	.09
July	645	210	429	26,378	.02	.02
August	252	69	142	8,732	.01	.01
September	325	69	155	9,223	.01	.01
October	252	124	190	11,683	.01	.01
November	475	210	310	18,446	.01	.01
December	645	325	403	24,780	.02	.02
The year	7,175	69	759	547,022	.03	.39
1899.[b]						
January	870	500	702	43,165	.027	.031
February	1,020	555	768	42,653	.030	.031
March	2,320	870	1,432	88,050	.055	.063
April	1,020	615	870	51,769	.034	.038
May	7,750	500	1,515	93,154	.059	.068
June	12,000	475	1,959	116,569	.076	.085
July	3,570	395	1,415	87,005	.055	.063
August	2,395	295	838	51,527	.032	.037

[a] Twentieth Ann. Rept. U. S. Geol. Survey, pt. 4, 1900, p. 318.
[b] Twenty-first Ann. Rept. U. S. Geol. Survey, pt. 4, 1901, p. 221.

Estimated monthly discharge of Republican River at Junction, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1899.						
September	345	100	187	11,127	0.007	0.008
October	1,140	35	302	18,569	.012	.014
November	395	63	175	10,413	.007	.008
December	800	205	488	30,006	.019	.022
The year	12,000	35	888	644,007	.034	.468
1900.[a]						
January	670	400	559	34,372	.02	.02
February	890	550	716	39,765	.03	.03
March	4,875	610	1,327	81,594	.05	.06
April	3,450	500	1,465	87,173	.06	.07
May	3,260	735	1,362	83,746	.05	.06
June	975	450	554	32,965	.02	.02
July	735	225	373	22,935	.01	.01
August	450	150	275	16,909	.01	.01
September	3,090	198	771	45,878	.03	.03
October	345	150	258	15,864	.01	.01
November	225	130	162	9,640	.01	.01
December	500	173	257	15,802	.01	.01
The year	4,875	130	673	486,643	.03	.34
1901.[b]						
January	570	405	510	31,359	.020	.023
February	1,015	450	669	37,154	.026	.027
March	1,750	850	1,030	63,332	.040	.046
April	5,010	930	1,530	91,041	.059	.066
May	1,015	405	747	45,931	.029	.033
June	510	315	406	24,159	.016	.018
July	450	75	177	10,883	.007	.008
August	110	20	53	3,259	.002	.002
September	3,160	20	906	53,911	.036	.040
October	770	405	564	34,679	.022	.025
November	510	405	472	28,086	.018	.020
December	660	450	594	36,524	.023	.027
The year	5,010	20	638	460,318	.025	.335

[a] Twenty-second Ann. Rept. U. S. Geol. Survey, pt. 4, 1902, p. 333.
[b] Water-Sup. and Irr. Paper No. 75, U. S. Geol. Survey, 1903, p. 141.

Estimated monthly discharge of Republican River at Junction, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1902.[a]						
January	945	600	710	43,656	0.03	0.03
February	1,425	735	775	43,041	.03	.03
March	1,480	570	887	54,589	.03	.03
April	1,207	510	654	38,916	.03	.03
May	7,800	482	2,128	130,846	.08	.09
June	8,400	1,260	3,338	198,625	.13	.15
July	23,100	840	7,887	484,953	.30	.35
August	11,700	260	2,046	125,804	.08	.09
September	10,500	88	1,646	97,944	.06	.07
October	5,150	427	1,436	88,296	.06	.07
November	787	400	471	28,026	.02	.02
December	2,443	220	422	25,944	.02	.02
The year	23,100	88	1,867	1,360,590	.07	.98
1903.						
January	1,102	325	684	42,058	.026	.030
February	3,600	482	1,217	67,589	.047	.049
March	13,500	1,155	5,099	313,525	.200	.230
April	1,102	375	624	37,130	.024	.027
May	47,520	682	12,112	744,738	.470	.540
June	44,280	1,370	8,162	485,673	.310	.340
July	9,750	997	2,578	158,515	.100	.120
August	10,350	735	3,254	200,081	.120	.140
September	3,020	400	859	51,114	.033	.037
October	1,315	240	466	28,653	.018	.021
November	1,207	455	746	44,390	.029	.032
December	1,102	570	757	46,546	.029	.033
The year	47,520	240	3,046	2,220,012	.117	1.599

[a] Water-Sup. and Irr. Paper No. 84, U. S. Geol. Survey, 1903, p. 102.

The hydrograph for this stream at this station is shown on Pl. XII, *C*. It is seen to resemble to some extent that for Kansas River at Lawrence, Kans. The minimum flow, however, occurs in the fall instead of in the winter. The maximum flow usually occurs some time between April 1 and July 31.

The monthly run-off per square mile prior to 1903 varied from 0.002

to 0.30 second-feet and the yearly run-off per square mile from 0.03 to 0.07 second-feet. The run-off for May, 1903, was 0.47 second-feet and the yearly run-off for 1903 was 0.117. The maximum monthly discharge is 230 times the minimum monthly discharge.

REPUBLICAN RIVER NEAR SUPERIOR, NEBR.

The station on Republican River near Superior, Nebr., is at a highway bridge, 150 feet upstream from a dam which diverts water into a mill race. The channel at this point is about 500 feet wide, broken by three piers. The bed is sand and silt and somewhat changeable. The velocity is sluggish at low stages. The first gage used at this place was an inclined timber fastened to the left bank just above the bridge. In the spring of 1898 two other gages were established here, one in the river a few feet upstream from the crest of the dam, the other in the mill race at a bridge about 150 feet below its head. The discharge of this mill race was measured at this bridge, and the difference between it and the river at the bridge gave the amount flowing over the dam. It was found, however, that the discharge of the mill race was regulated more or less by the mill below, so that there was no relation between the gage heights and the discharge. The method of remedying this defect is described on page 103 of Water-Supply and Irrigation Paper No. 84.

The run-off data obtained at this station are given in the following table:

Estimated monthly discharge of Republican River near Superior, Nebr.

[Drainage area, 22,347 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1896.[a]						
June[b]			627	13,244	0.028	0.011
July[c]			1,179	70,155	.093	.059
August			659	40,520	.029	.033
September[d]			279	16,048	.012	.013
October			230	14,142	.010	.012
November[e]			449	25,827	.020	.022

[a] Based on daily discharges published in Eighteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1897, p. 202. Discharge at both dam and mill race are included.
[b] For 11 days.
[c] For 30 days.
[d] For 29 days.
[e] For 29 days.

Estimated monthly discharge of Republican River near Superior, Nebr.—Cont'd.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1897.[a]						
January [b]	900	555	710	43,656	0.032	0.037
February [b]	1,520	420	906	50,316	.041	.043
March [b]	650	385	510	31,359	.023	.026
April	2,870	620	1,244	74,023	.056	.062
May	1,130	180	420	25,825	.019	.022
June [b]	750	180	359	21,362	.016	.018
July	4,310	119	922	56,690	.041	.047
August	1,460	94	259	15,924	.012	.014
September	241	0	71	4,225	.003	.003
1898.[c]						
April [d]	2,130	490	755	44,926	.03	.03
May	1,870	500	1,053	64,747	.05	.06
June	2,300	560	1,235	73,487	.06	.07
July	1,345	220	479	29,453	.02	.02
August	380	100	195	11,990	.01	.01
September	470	185	289	17,197	.01	.01
October	365	145	264	16,233	.01	.01
November [e]	470	320	382	22,731	.02	.02
1899.[f]						
March [g]	956	601	706	43,411	.031	.036
April	842	375	578	34,398	.026	.029
May	1,800	292	511	31,420	.023	.026
June	4,036	141	531	31,597	.021	.023
July	1,384	209	458	28,162	.020	.023
August	868	136	303	18,631	.014	.016
September	102	6	50	2,975	.002	.002
October	101	24	53	3,259	.002	.002
1900.[h]						
April	1,671	565	909	54,089	.041	.046
May	2,945	320	735	45,193	.033	.038
June	674	250	392	23,326	.018	.020

[a] Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1899, p. 339. Discharges represent amount going over dam only; the mill race carries on an average, 85 second-feet.
[b] Approximate.
[c] Twentieth Ann. Rept. U. S. Geol. Survey, pt. 4, 1900, p. 317.
[d] For entire month; based on gage heights of April 20 to 30.
[e] For entire month; based on gage heights of November 1 to 19.
[f] Twenty-first Ann. Rept. U. S. Geol. Survey, pt. 4, 1901, p. 220.
[g] For entire month, based on gage heights of March 19 to 31.
[h] Twenty-second Ann. Rept. U. S. Geol. Survey, pt. 4, 1902, p. 332.

Estimated monthly discharge of Republican River near Superior, Nebr.—Cont'd.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1900.						
July	349	67	159	9,777	0.006	0.007
August	1,278	24	149	9,162	.006	.007
September	2,010	8	156	9,283	.006	.007
October	148	26	69	4,243	.003	.003
November [a]	162	36	87	5,177	.004	.004
1901. [b]						
April	1,507	637	961	57,183	.043	.048
May	622	146	364	22,381	.016	.018
June	480	117	219	13,031	.010	.011
July	154	5	44	2,618	.002	.002
August	856	5	70	4,304	.003	.003
September	4,991	10	1,246	74,142	.056	.062
October	1,490	285	443	27,239	.020	.020
November	1,030	312	419	24,932	.019	.021
1902. [c]						
March 9 to 31	.....	.....	675	29,453	.03	.02
April	740	390	555	33,020	.02	.02
May	2,810	335	1,269	78,020	.06	.07
June	3,235	320	1,372	81,640	.06	.07
July [d]	12,490	970	3,080	189,360	.14	.16
August	1,920	330	674	41,440	.03	.03
September	10,875	155	1,175	69,910	.05	.06
October	1,995	520	937	57,610	.04	.05
November	970	460	622	37,010	.03	.03
1903.						
March 14–31	7,750	1,260	2,692	96,120	.120	.080
April	1,260	705	918	54,625	.041	.046
May	14,100	845	4,491	276,141	.201	.232
June	6,455	835	1,921	114,307	.086	.096
July	7,825	465	1,948	119,778	.087	.100
August	3,285	520	1,230	75,630	.055	.063
September	700	270	389	23,147	.017	.019
October	390	245	326	20,045	.015	.017
November	690	130	417	24,813	.019	.021

[a] For entire month, based on gage heights of November 1 to 24.
[b] Water-Sup. and Irr. Paper No. 75, U. S. Geol. Survey, 1903, p. 140.
[c] Water-Sup. and Irr. Paper No. 84, U. S. Geol. Survey, 1903, p. 105.
[d] Exclusive of four days flood discharge.

It is seen that the minimum monthly discharge occurs in September and is less than 50 second-feet. The smallest daily discharge is 0. The maximum monthly discharge is 4,491 second-feet, which is 102 times the minimum monthly discharge. The monthly run-off per square mile varies from 0.002 to 0.20 second-feet.

REPUBLICAN RIVER NEAR BENKELMAN, NEBR.

A station was established on North Fork of Republican River about 2 miles west of Benkelman, Nebr., November 4, 1894. A station was also established on South Fork near Benkelman, at about the same time. The former station was maintained continuously until September 1, 1895. The other was maintained, with short breaks, to September 31, 1895. The stream bed at these stations is sandy and shifting, and each flood changes the channel by a considerable amount, so that a rating curve at either station was applicable for only a few months. Stations were again established at these places in May, 1903.

The monthly discharge was as follows:

Estimated monthly discharge of Republican River at Benkelman, Nebr.

Month.	Discharge in second-feet.			Total in acre-feet.
	Maximum.	Minimum.	Mean.	
NORTH FORK.				
1903.				
May 20–31	71	47	58	1,382
June	81	43	59	5,511
July	112	19	39	2,398
August	61	26	32	1,968
September	47	19	39	2,321
October	54	43	49	3,013
November 1–20[a]	64	50	56	2,204
SOUTH FORK.				
1903.				
May 20–31	57	36	48	1,148
June	65	7	37	2,202
July	36	7	15	922
August	79	7	25	1,537
September[b]	22	7	11	655
October	50	22	39	2,398
November 1–20[a]	65	50	57	2,269

[a] Frozen November 21 to December 31.
[b] Records from 6 to 13, *inclusive*, are missing.

SOLOMON RIVER AT BELOIT AND NEAR NILES, KANS.

A station was established on Solomon River at a highway bridge in Beloit, April 22, 1895, and was maintained continuously until June 30, 1897, when it was discontinued on account of poor section. The present gaging station on Solomon River is located 56 miles down the river from Beloit, at an iron bridge about 1½ miles from Niles. It was established May 5, 1897, and has been maintained continuously since that date. The channel at the present station is about 150 feet wide, nearly straight for 100 feet above and below, and broken by two piers. The bed is clay and silt. The velocity is sluggish at low stages. Surface fluctuations are measured with a wire gage.

The run-off data obtained at these two stations on Solomon River are given in the following table:

Estimated monthly discharge of Solomon River at Beloit, Kans.

[Drainage area, 5,539 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1895.[a]						
July	[b]21,000	159	2,202	135,396	0.399	0.460
August	[b]24,000	172	1,629	100,164	.294	.339
September	615	14	156	9,283	.028	.032
October	62	7	28	1,722	.005	.006
November	26	7	23	1,369	.004	.005
December	250	7	114	7,010	.021	.024
1896.[a]						
January	121	5	88	5,411	.016	.018
February	121	19	103	5,925	.018	.020
March	123	8	94	5,780	.017	.019
April	[b]18,500	8	1,658	98,658	.300	.334
May	4,700	72	429	26,378	.077	.089
June	8,740	104	852	50,698	.154	.172
July	[b]21,800	108	1,238	76,122	.226	.255
August	803	92	354	21,767	.065	.073
September	960	28	130	7,735	.023	.026
October	6,760	10	302	18,569	.055	.062
November	117	34	76	4,522	.013	.015
December	124	38	92	5,657	.017	.019
The year	[b]21,800	5	451	327,222	.083	11.02

[a] Eighteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1897, pp. 209-210.
[b] Estimated. NOTE.—Recent measurements show that these estimates are too large.

Estimated monthly discharge of Solomon River at Beloit, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1897.[a]						
January	225	16	130	8,024	0.023	0.027
February	387	20	193	10,724	.035	.036
March	312	92	227	13,933	.041	.047
April	7,470	180	663	39,463	.120	.133
May	368	18	150	9,254	.027	.031
June	6,400	19	1,377	81,913	.249	.278

[a] Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1899, p. 341.

Estimated monthly discharge of Solomon River near Niles, Kans.

[Drainage area, 6,815 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1897.[a]						
May[b]	669	161	267	16,411	0.039	0.045
June	4,500	103	1,083	64,433	.159	.177
July	5,380	215	838	51,551	.123	.142
August	320	108	193	11,892	.028	.033
September	200	65	106	6,319	.016	.017
October	142	62	97	5,976	.014	.016
November	154	86	114	6,784	.017	.019
December	161	60	100	6,124	.015	.017
1898.[c]						
January	172	126	155	9,531	.02	.02
February	260	109	164	9,108	.02	.02
March	190	98	131	8,055	.02	.02
April	353	109	146	8,688	.02	.02
May	677	160	243	14,942	.04	.04
June	1,175	196	534	31,775	.08	.09
July	335	79	156	9,592	.02	.02
August	103	35	56	3,443	.01	.01
September	190	42	79	4,701	.01	.01

[a] Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1899, p. 343.
[b] Estimated from gage heights of May 6 to 31.
[c] Twentieth Ann. Rept. U. S. Geol. Survey, pt. 4, 1900, p. 314.

Estimated monthly discharge of Solomon River near Niles, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1898.						
October	79	47	58	3,566	0.01	0.01
November	120	47	72	4,284	.01	.01
December	268	87	147	9,039	.02	.02
The year	1,175	35	162	116,724	.02	.29
1899.[a]						
January	303	131	178	10,944	.026	.030
February	202	147	178	9,886	.026	.027
March	315	125	177	10,883	.026	.030
April	159	90	128	7,617	.019	.021
May	2,364	85	304	18,692	.045	.052
June	6,240	131	1,789	106,453	.262	.29
July	8,895	147	580	35,663	.085	.098
August	1,283	105	272	16,725	.039	.045
September	131	53	88	5,236	.013	.014
October	1,152	37	126	7,747	.018	.021
November	120	53	81	4,820	.012	.013
December	131	62	99	6,087	.015	.017
The year	6,240	37	328	236,880	.048	.645
1900.[b]						
January	128	58	90	5,534	.01	.01
February	159	74	108	5,720	.02	.02
March	1,870	80	246	15,126	.04	.05
April	4,627	74	760	45,228	.11	.12
May	1,156	211	532	32,711	.08	.09
June	860	80	230	13,686	.03	.03
July	630	54	135	8,301	.02	.02
August	860	66	242	14,880	.04	.05
September	2,020	54	294	17,494	.04	.04
October	755	66	116	7,133	.02	.02
November	100	54	71	4,225	.01	.01
December	100	58	75	4,612	.01	.01
The year	4,627	54	241	174,645	.04	.47

[a] Twenty-first Ann. Rept. U. S. Geol. Survey, pt. 4, 1901, p. 223.
[b] Twenty-second Ann. Rept. U. S. Geol. Survey, pt. 4, 1902, p. 334.

Estimated monthly discharge of Solomon River near Niles, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1901.[a]						
January	114	46	91	5,595	0.013	0.015
February	193	54	90	4,998	.013	.014
March	341	100	152	9,346	.022	.025
April	2,020	114	452	26,896	.066	.074
May	303	89	142	8,731	.021	.024
June	989	46	157	9,342	.023	.026
July	54	7	27	1,660	.004	.005
August	62	7	18	1,107	.003	.003
September	1,604	70	243	14,460	.036	.040
October	89	38	67	4,120	.010	.012
November	176	38	99	5,891	.015	.017
December	100	54	79	4,858	.012	.014
The year	2,020	7	135	97,004	.020	.269
1902.[b]						
January	152	94	106	6,517	.02	.02
February	142	102	119	6,608	.02	.02
March	207	38	101	6,210	.02	.02
April	152	54	89	5,296	.01	.01
May	1,415	38	285	17,524	.04	.05
June	6,155	316	1,392	82,830	.20	.22
July	7,040	252	2,492	153,227	.37	.43
August	2,375	299	847	52,080	.12	.14
September	7,040	132	1,392	82,830	.20	.22
October	7,780	761	2,263	139,145	.33	.38
November	855	470	601	35,764	.09	.10
December	490	316	402	24,718	.06	.07
The year	7,780	38	841	612,749	.12	1.68
1903.						
January	410	282	341	20,967	.050	.058
February	830	237	356	19,882	.052	.054
March	5,002	587	1,866	114,736	.270	.310
April	564	120	345	20,529	.051	.057
May	9,946	635	4,835	297,292	.710	.820
June	10,602	1,765	4,447	264,615	.650	.720

[a] Water-Sup. and Irr. Paper No. 75, U. S. Geol. Survey, 1903, p. 142.
[b] Water-Sup. and Irr. Paper No. 84, U. S. Geol. Survey, 1903, p. 107.

Estimated monthly discharge of Solomon River near Niles, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1903.						
July	4,074	835	1,658	101,946	0.240	0.280
August	7,091	1,190	3,625	222,893	.530	.610
September	2,460	472	970	57,719	.140	.160
October	495	363	427	26,255	.063	.073
November	760	343	456	27,134	.067	.075
The period	10,602	120	1,611	1,173,958	.235	3.217

The hydrographs for this stream at these stations are shown in Pl. XIII, *B*, p. 64. The minimum monthly flow was very small, and less than 20 second-feet in August, 1901. The flood flow is occasionally very large, and was 4,835 second-feet in May, 1903. The maximum monthly discharge is 269 times the minimum monthly discharge. The monthly run-off per square mile varies from 0.003 to 0.710.

SALINE RIVER AT BEVERLY AND NEAR SALINA, KANS.

A station was established on Saline River at Beverly, April 18, 1895, and was maintained continuously from that date to June 30, 1897, when it was discontinued on account of poor section. The present station on Saline River is located at a highway bridge, about 4½ miles north of Salina and 24 miles down the river from Beverly. It was established May 4, 1897, to take the place of the station at Beverly, and has been maintained continuously from that time to the present. The channel is about 170 feet wide, nearly straight for a short distance above and below the bridge, and without piers. The bed is soft material; the velocity sluggish. Surface fluctuations are measured with a wire gage. Both banks are subject to overflow during floods.

The run-off results obtained at this station are given in the following tables:

Estimated monthly discharge of Saline River at Beverly, Kans.

[Drainage area, 2,730 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1895.[a]						
July	[b]10,000	73	574	35,294	0.201	0.232
August	339	58	101	6,210	.037	.043
September	92	9	38	2,261	.014	.016
October	58	6	27	1,660	.010	.012
November	62	9	33	1,964	.012	.013
December	57	13	33	2,029	.012	.014
1896.[a]						
January	76	26	52	3,197	.019	.022
February	96	33	51	2,934	.018	.020
March	65	33	49	3,013	.018	.020
April	693	37	151	8,986	.055	.062
May	3,000	45	313	19,245	.114	.132
June	[b]16,000	167	2,291	136,324	.839	.936
July	679	103	286	17,586	.104	.120
August	493	41	107	6,579	.037	.045
September	90	33	58	3,451	.021	.024
October	[b]6,130	31	260	15,987	.095	.109
November	188	41	75	4,463	.027	.031
December	98	31	62	3,505	.023	.027
The year	[b]16,000	26	313	225,270	.114	1.548
1897.[c]						
January	108	17	48	2,927	.017	.020
February	243	27	73	4,054	026	.027
March	67	32	46	2,835	.017	.019
April	626	49	192	11,424	.070	.078
May	357	53	146	8,953	.053	.061
June	[b]11,000	38	452	26,920	.166	.185

[a] Eighteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1897, p. 212.
[b] Approximate.
[c] Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1899, p. 344.

Estimated monthly discharge of Saline River near Salina, Kans.

[Drainage area, 3,311 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second feet per square mile.	Depth in inches.
1897.[a]						
May	335	79	147	9,039	0.044	0.051
June	[b] 4,400	51	256	15,203	.077	.085
July	[a] 685	36	83	5,104	.025	.028
August	381	28	74	4,532	.022	.025
September	57	20	30	1,803	.009	.010
October	70	16	32	1,955	.010	.011
November	48	26	38	2,279	.012	.013
December	46	28	41	2,521	.012	.014
1898.[c]						
January	82	40	57	3,505	.02	.02
February	94	34	50	2,777	.02	.02
March	46	24	37	2,275	.01	.01
April	60	24	35	2,083	.01	.01
May	1,095	34	109	6,702	.03	.04
June	1,100	72	271	16,126	.08	.09
July	421	22	86	5,288	.03	.03
August	32	7	20	1,230	.01	.01
September	970	6	121	7,440	.04	.04
October	52	22	35	2,152	.01	.01
November	40	15	25	1,488	.01	.01
December	690	29	144	8,854	.04	.04
The year	1,100	6	83	59,920	.03	.33
1899.[d]						
January	110	60	67	4,120	.020	.023
February	102	50	63	3,499	.019	.020
March	117	50	80	4,919	.024	.028
April	72	30	53	3,154	.016	.018
May	1,010	18	155	9,531	.047	.054
June	5,000	110	1,334	79,378	.403	.45
July	684	132	279	17,155	.084	.097
August	152	62	100	6,149	.030	.035
September	102	55	68	4,046	.021	.023

[a] Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1899, p. 346
[b] Approximate.
[c] Twentieth Ann. Rept. U. S. Geol. Survey, pt. 4, 1900, p. 316.
[d] Twenty-first Ann. Rept. U. S. Geol. Survey, pt. 4, 1901, p. 224.

Estimated monthly discharge of Saline River near Salina, Kans.—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1899.						
October	386	40	79	4,858	0.024	0.028
November	181	70	88	5,236	.027	.030
December	102	70	84	5,165	.025	.029
The year	5,000	18	204	147,210	.062	.835
1900.[a]						
January	55	42	49	3,013	.01	.01
February	55	42	47	2,610	.01	.01
March	700	44	114	7,010	.03	.03
April	2,930	50	471	28,026	.04	.06
May	1,100	166	347	21,336	.10	.12
June	1,427	70	245	14,578	.07	.08
July	3,370	46	382	23,488	.12	.14
August	370	36	76	4,678	.02	.02
September	1,160	40	120	7,141	.04	.04
October	133	40	56	3,443	.02	.02
November	76	46	51	3,035	.02	.02
December	60	46	53	3,259	.02	.02
The year	3,370	36	168	121,612	.05	.67
1901.[b]						
January	71	47	56	3,443	.017	.020
February	115	55	74	4,110	.022	.023
March	122	51	76	4,673	.023	.027
April	3,580	85	470	27,967	.142	.158
May	315	60	112	6,887	.034	.039
June	2,680	37	376	22,374	.114	.127
July	51	22	31	1,906	.009	.010
August	153	25	47	2,890	.014	.016
September	85	40	53	3,154	.016	.018
October	60	37	47	2,890	.014	.016
November	65	40	53	3,154	.016	.018
December	65	47	57	3,505	.017	.020
The year	3,580	22	121	86,953	.036	.492

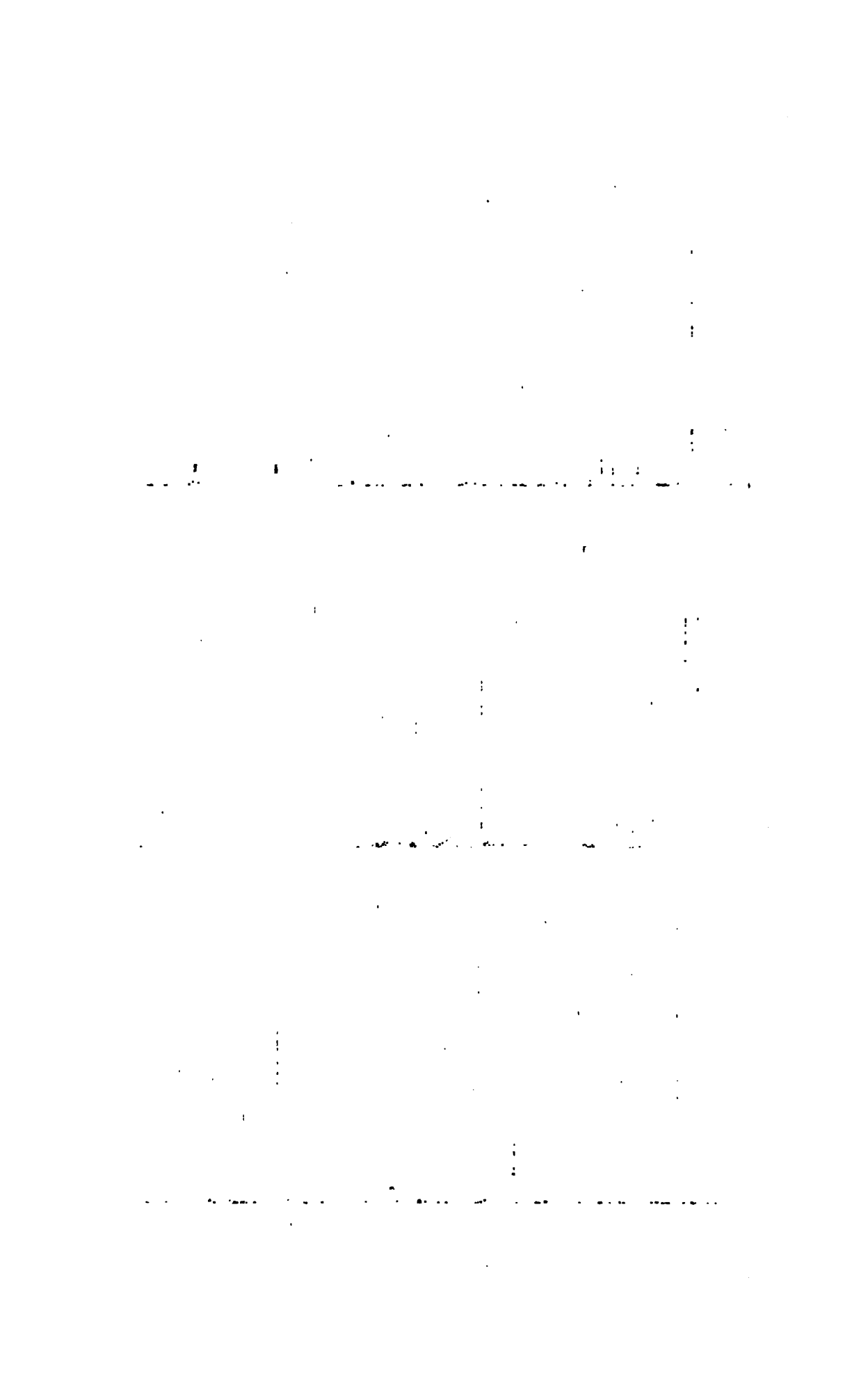
[a] Twenty-second Ann. Rept. U. S. Geol. Survey, pt. 4, 1902, p. 335.
[b] Water-Sup. and Irr. Paper No. 75, U. S. Geol. Survey, 1903, p. 142.

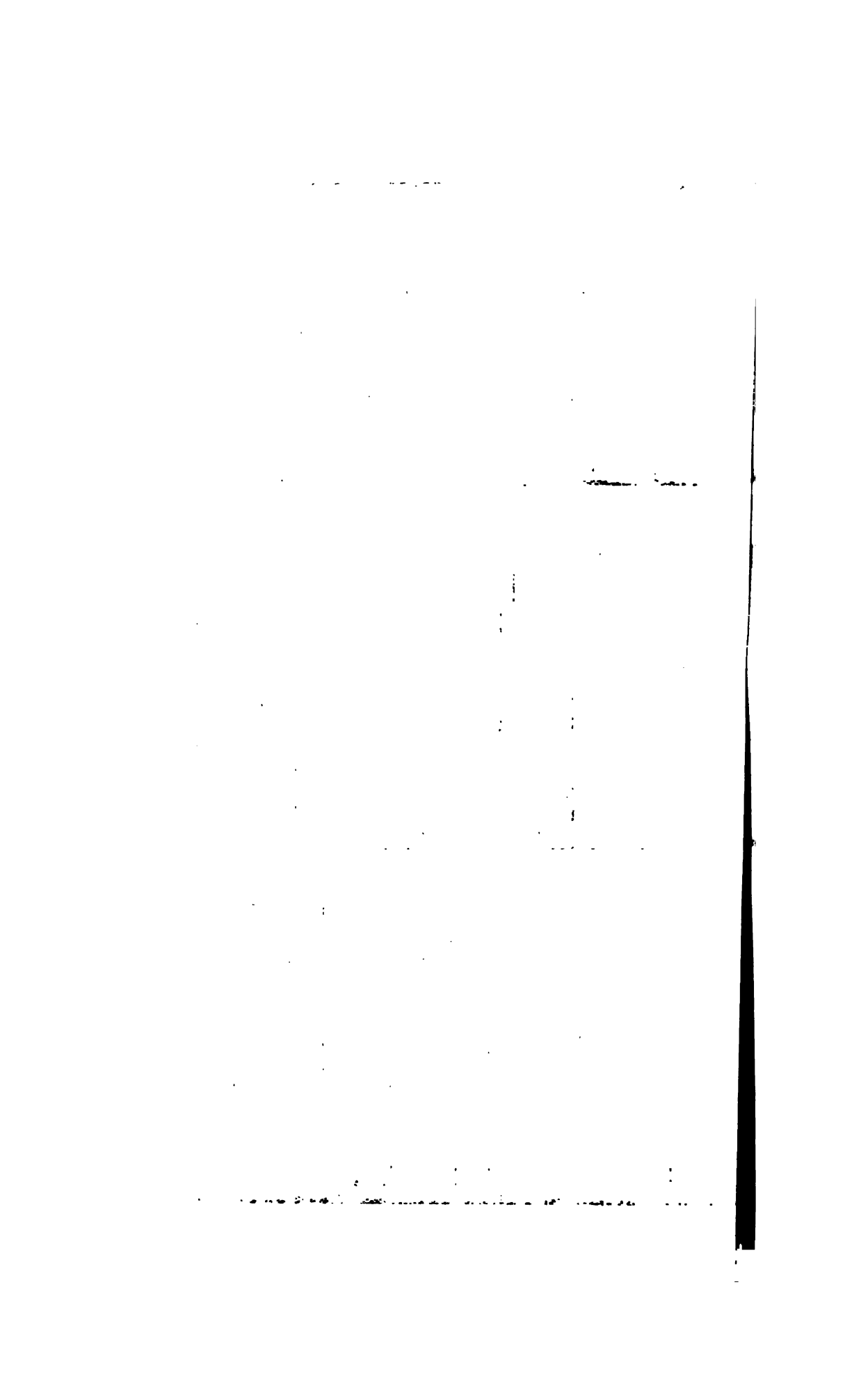
Estimated monthly discharge of Saline River near Salina, Kans.—Continued.

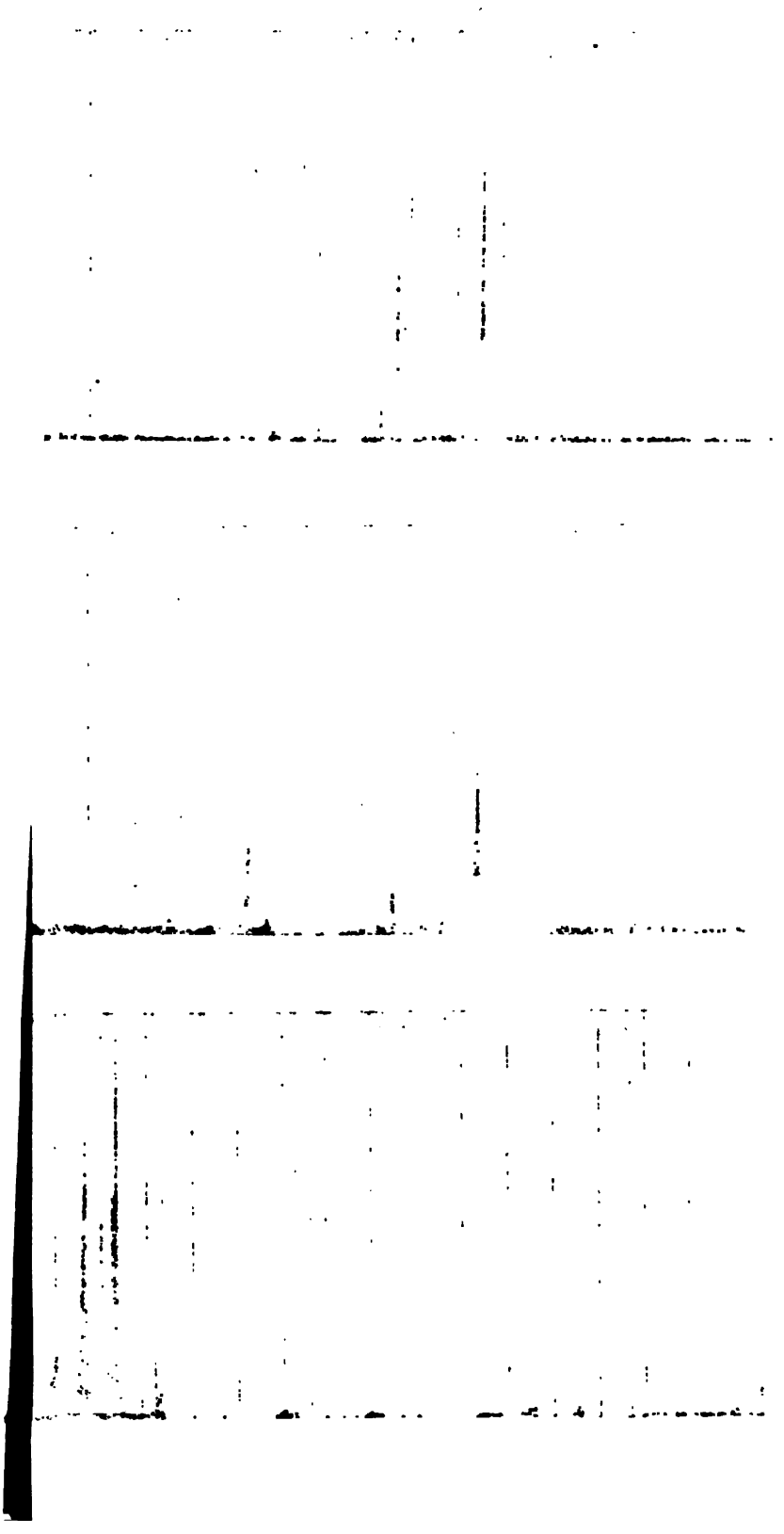
Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1902.[a]						
January	90	45	64	3,935	0.019	0.022
February	70	41	60	3,332	.018	.019
March	75	34	45	2,767	.014	.016
April	41	25	35	2,083	.011	.012
May	698	25	112	6,886	.034	.039
June	1,892	70	792	47,127	.239	.267
July	2,507	110	621	38,184	.188	.217
August	1,910	100	512	31,482	.155	.179
September	3,920	110	874	52,007	.264	.295
October	3,920	364	1,089	66,960	.329	.379
November	424	250	305	18,159	.092	.103
December	275	190	234	14,388	.071	.082
The year	3,920	25	395	287,310	.120	1.639
1903.						
January	240	180	205	12,605	.062	.071
February	280	140	207	11,496	.063	.064
March	1,336	210	628	38,614	.190	.220
April	340	220	271	16,126	.082	.091
May	7,580	910	2,685	165,094	.810	.930
June	7,895	835	2,873	170,955	.870	.970
July	985	332	533	32,773	.160	.184
August	3,410	458	1,434	88,173	.430	.500
September	472	207	322	19,799	.097	.108
October	262	168	212	13,035	.064	.074
November	304	168	223	13,270	.067	.075

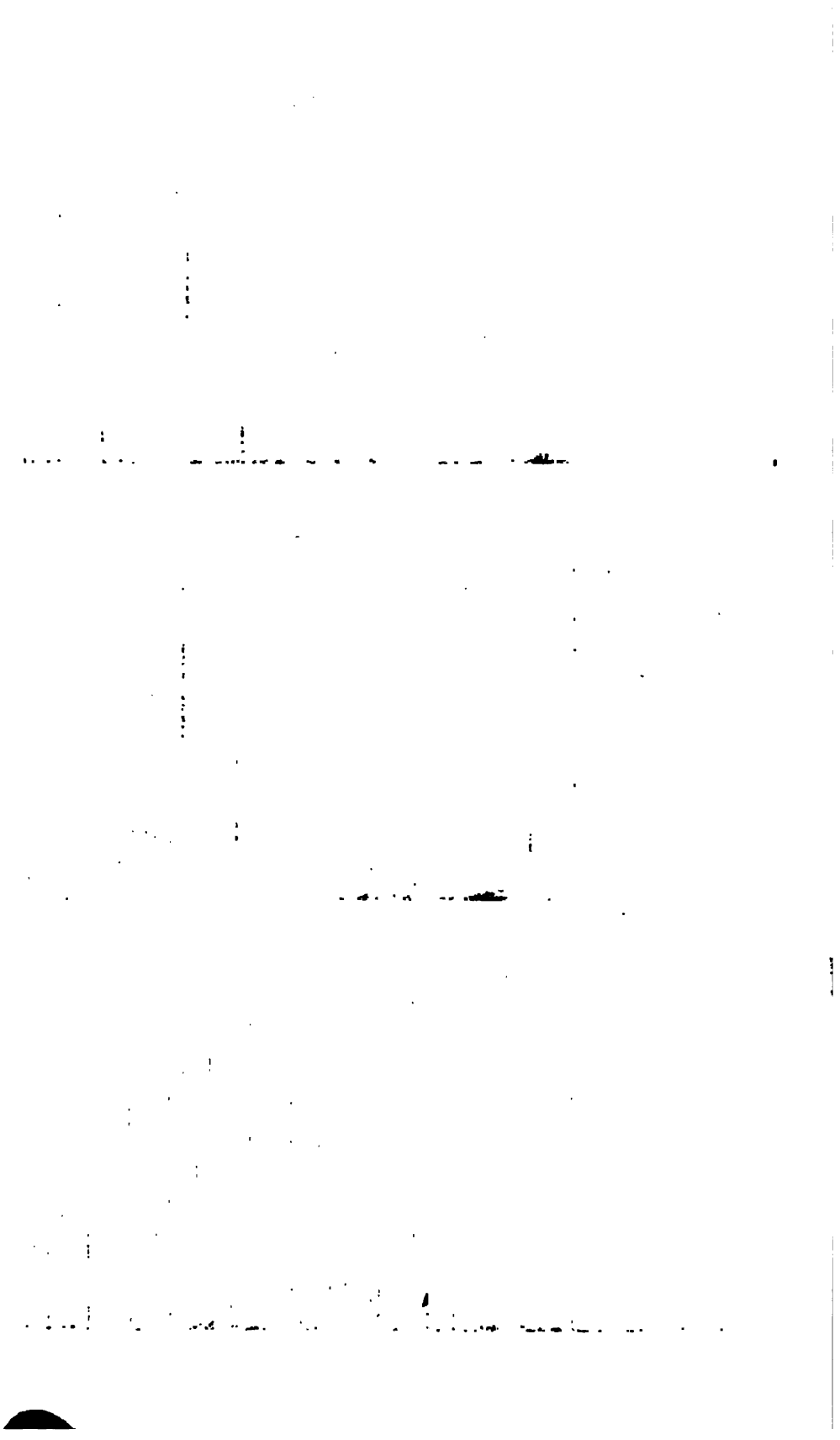
[a] Water-Sup. and Irr. Paper No. 84, U. S. Geol. Survey, 1903, p. 110.

The hydrograph for this stream at this station is shown on Pl. XIII, *A*. The mean monthly flow is seen to be very small, being only 20 second-feet in August, 1898. The maximum is very large, and usually occurs between May 1 and July 31. The monthly run-off per square mile varies from less than 0.01 to 0.87 second-foot. The yearly run-off per square mile varies from 0.03 to 0.120 second-foot for the years with complete record. The maximum monthly discharge is 143 times the minimum monthly discharge.









SMOKY HILL RIVER AT ELLSWORTH, KANS.

The station on Smoky Hill River is at a highway bridge in Ellsworth. It was established April 17, 1895, and has been maintained continuously from that time to the present. The channel is about 180 feet wide at ordinary stage and nearly straight for 200 feet above and below the station. The bed is mainly sandy material and somewhat changeable. Velocity is moderate at ordinary stages. The surface fluctuations are read from the vertical gage bolted to one of the bridge piers.

The run-off results obtained at this station are given in the following table:

Estimated monthly discharge of Smoky Hill River at Ellsworth, Kans.

[Drainage area, 7,980 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1895.[a]						
April 17–30	14	11	12	336	0.002	0.001
May	20	9	11	676	.001	.001
June	4,900	10	690	41,058	.086	.096
July	21,000	202	1,438	88,420	.180	.208
August	344	65	186	11,437	.023	.027
September	135	19	44	2,618	.006	.007
October	22	14	18	1,107	.002	.003
November	19	12	18	1,071	.002	.003
December	22	11	17	1,045	.002	.003
1896.[a]						
January	57	17	25	1,537	.003	.004
February	49	18	26	1,496	.003	.004
March	21	14	18	1,107	.002	.002
April	800	14	130	7,736	.016	.018
May	116	18	62	3,812	.008	.009
June	1,713	72	313	18,625	.039	.044
July	951	119	230	14,142	.029	.033
August	372	66	115	7,071	.014	.016
September	117	50	72	4,284	.009	.010
October	255	25	54	3,320	.007	.008
November	122	20	54	3,213	.007	.008
December	43	18	34	2,337	.004	.005
The year	1,713	14	95	68,680	.012	.161

[a] Eighteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1897, p. 214.

Estimated monthly discharge of Smoky Hill River at Ellsworth, Kans.—Cont'd.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches
1897.[a]						
January	153	22	68	4,198	0.008	0.010
February	213	35	122	6,798	.015	.016
March	77	22	35	2,164	.004	.005
April	385	35	158	9,401	.020	.022
May	385	87	173	10,624	.022	.025
June	240	35	87	5,153	.011	.012
July	97	19	38	2,336	.005	.006
August	295	15	95	5,854	.012	.014
September	130	15	40	2,386	.005	.006
October	97	10	32	1,949	.004	.005
November	77	15	38	2,285	.005	.005
December	107	19	38	2,337	.005	.006
The year	385	10	77	55,485	.010	.132
1898.[b]						
January	70	18	36	2,214	.005	.005
February	95	21	47	2,610	.006	.006
March	32	16	24	1,476	.003	.003
April	37	15	20	1,190	.003	.003
May	1,000	32	136	8,362	.017	.020
June	717	78	281	16,721	.035	.039
July	78	18	43	2,644	.005	.006
August	105	15	25	1,537	.003	.004
September	763	14	131	7,795	.016	.018
October	55	18	30	1,845	.004	.004
November	42	18	19	1,131	.002	.003
December	147	24	53	3,259	.006	.007
The year	1,000	14	70	50,784	.009	.118
1899.[c]						
January	75	14	44	2,705	.0055	.0063
February	84	42	63	3,499	.0079	.0082
March	75	19	40	2,460	.0050	.0058
April	35	10	20	1,190	.0025	.0028
May	23	10	11	676	.0014	.0016

[a] Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 4, 1897, p. 347.
[b] Twentieth Ann. Rept. U. S. Geol. Survey, pt. 4, 1900, p. 315.
[c] Twenty-first Ann. Rept. U. S. Geol. Survey, pt. 4, 1901, p. 225.

Estimated monthly discharge of Smoky Hill River at Ellsworth, Kans.—Cont'd.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1899.						
June	3,890	10	492	29,276	0.0617	0.0688
July	7,947	56	832	51,158	.1043	.1202
August	862	42	152	9,346	.0190	.0219
September	75	19	46	2,737	.0058	.0064
October	124	14	48	2,951	.0060	.0069
November	42	19	30	1,785	.0038	.0043
December	35	19	27	1,660	.0034	.0039
The year	7,947	10	150	109,443	.0189	.2571
1900.[a]						
January[b]	97	40	49	3,013	.006	.007
February[b]	.....	.....	60	3,332	.008	.008
March[b]	183	40	66	4,058	.008	.009
April	594	35	156	9,283	.020	.022
May	660	77	147	9,039	.018	.021
June	616	87	241	14,340	.030	.033
July	705	35	117	7,194	.015	.017
August	97	18	47	2,890	.006	.007
September	616	18	140	8,331	.018	.020
October	144	40	78	4,796	.010	.012
November	40	22	28	1,666	.004	.004
December	26	18	21	1,291	.003	.003
The year	.....	.....	96	69,233	.012	.163
1901.[c]						
January	120	18	50	3,074	.006	.007
February	165	20	72	3,999	.009	.009
March	50	20	33	2,029	.004	.005
April	662	23	124	7,378	.016	.018
May	81	23	47	2,890	.006	.007
June	142	23	48	2,856	.006	.007
July	18	5	9	553	.001	.001
August	142	5	46	2,828	.006	.007
September	142	12	62	3,689	.008	.009

[a] Twenty-second Ann. Rept. U. S. Geol. Survey, pt. 4, 1902, p. 336.
[b] Approximate.
[c] Water-Sup. and Irr. Paper No. 75, U. S. Geol. Survey, 1903, p. 143.

Estimated monthly discharge of Smoky Hill River at Ellsworth, Kans.—Cont'd.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.
1901.						
October	73	14	36	2,214	0.005	0.006
November	14	8	11	655	.001	.001
December	50	12	22	1,353	.003	.003
The year	652	5	47	33,518	.006	.080
1902.[a]						
January	55	17	24	1,475	.003	.003
February	100	17	38	2,110	.005	.005
March	290	19	39	2,398	.005	.006
April	23	15	18	1,071	.002	.002
May	700	13	87	5,349	.011	.013
June	4,856	40	821	48,853	.103	.115
July	3,267	137	891	54,785	.112	.129
August	7,375	62	1,021	62,779	.128	.148
September	7,840	100	1,033	61,468	.130	.144
October	1,390	187	588	36,155	.074	.085
November	187	62	125	7,438	.016	.017
December	85	40	64	3,935	.008	.009
The year	7,840	13	396	287,816	.050	.676
1903.						
January	131	50	69	4,243	.0086	.0099
February	110	50	65	3,610	.0081	.0084
March	1,410	62	431	26,501	.0340	.0620
April	1,834	62	182	10,830	.0230	.0260
May	11,392	510	2,356	144,865	.3000	.3400
June	3,750	342	937	55,755	.1200	.1300
July	305	120	180	11,068	.2300	.2600
August	2,708	155	735	45,193	.0920	.1100
September	143	50	81	4,820	.0100	.0110
October	92	45	58	3,566	.0073	.0084
November	120	45	70	4,304	.0088	.0098
December	92	36	44	2,705	.0055	.0063
The year	11,392	36	434	317,460	.0723	.9818

[a] Water-Sup. and Irr. Paper No. 84, U. S. Geol. Survey, 1903, p. 112.

The hydrograph of this stream at this station is shown on Pl. XIII, *C*. The minimum monthly flow is seen to have been less than 10 second-feet during July, 1901. The maximum discharge was 1,438 second-feet in July, 1895, which is 160 times the minimum monthly discharge. The monthly run-off per square mile varies from less than 0.001 to 0.18, and the yearly run-off from 0.006 to 0.050 second-feet.

COMPARISON OF FLOOD AND ORDINARY DATA.

In the following table is given a comparison of the ordinary low flow with the flood flow of these streams. The flood flow is seen to be many times larger than the ordinary low-water flow.

Low-water and flood-flow data of streams in Kansas River basin.

Stream and place.	Low-water data.		
	Date.	Gage.	Discharge.
	1902.		
Kansas, Lecompton, Kans	April	3.00 to 3.40	2,375 to 3,415
Blue, Manhattan, Kans	do	3.80 4.20	480 560
Republican, Junction, Kans	do	3.60 4.30	510 1,155
Solomon, Niles, Kans	do	4.40 5.50	54 152
Saline, Salina, Kans	do	3.10 3.60	25 41
Smoky Hill, Ellsworth, Kans	do	.75 .95	15 23

Stream and place.	Flood-flow data.		
	Date.	Gage.	Discharge.
	1903.		
Kansas, Lecompton, Kans	May 31	29.5	233,070
Blue, Manhattan, Kans	do	36.5	86,640
Republican, Junction, Kans	May 30	18.2	59,400
Solomon, Niles, Kans	June 1	32.9	12,000
Saline, Salina, Kans	do	33.9	11,200
Smoky Hill, Ellsworth, Kans	May 29	16.0	11,610

The following table gives a comparison of the discharges at the six gaging stations and also the run-off for four months of 1903. The stations are arranged in order from the west to the east. The increase in run-off from the west to the east is clearly shown.

Comparisons of monthly discharge and run-off at gaging stations in Kansas River watershed.

MONTH OF APRIL, 1902.

Stream and place.	Discharge in second-feet.				Run-off.	
	Maximum.	Minimum.	Mean.	Total in acre-feet.	Second-feet per square mile.	Depth in inches.
Smoky Hill, Ellsworth, Kans	23	15	18	1,071	0.002	0.002
Saline, Salina, Kans	41	25	35	2,083	.01	.01
Solomon, Niles, Kans	152	54	89	5,296	.01	.01
Republican, Junction, Kans	1,207	510	654	38,916	.03	.03
Republican, Superior, Nebr	740	390	555	33,020	.02	.02
Blue, Manhattan, Kans	560	408	483	28,740	.05	.06
Kansas, Lecompton, Kans	3,415	2,250	2,653	157,864	.05	.06

MONTH OF MAY, 1902.

Smoky Hill, Ellsworth, Kans	700	13	87	5,349	0.011	0.013
Saline, Salina, Kans	698	25	112	6,886	.034	.039
Solomon, Niles, Kans	1,415	38	285	17,524	.04	.05
Republican, Junction, Kans	7,800	482	2,128	130,846	.08	.10
Republican, Superior, Nebr	2,810	335	1,269	78,020	.06	.07
Blue, Manhattan, Kans	17,510	408	2,157	132,629	.23	.27
Kansas, Lecompton, Kans	23,075	2,125	8,435	518,648	.14	.16

MONTH OF JUNE, 1902.

Smoky Hill, Ellsworth, Kans	4,856	40	821	48,853	0.10	0.12
Saline, Salina, Kans	1,892	70	792	47,127	.24	.27
Solomon, Niles, Kans	6,155	316	1,392	82,890	.20	.22
Republican, Junction, Kans	8,400	1,260	3,338	198,625	.13	.15
Republican, Superior, Nebr	3,235	320	1,372	81,640	.06	.07
Blue, Manhattan, Kans	17,510	910	6,298	374,757	.66	.74
Kansas, Lecompton, Kans	28,500	11,110	17,131	1,019,365	.29	.329

MONTH OF JULY, 1902.

Smoky Hill, Ellsworth, Kans	3,267	137	891	54,785	0.112	0.129
Saline, Salina, Kans	2,507	110	621	38,184	.188	.217
Solomon, Niles, Kans	7,040	252	2,492	153,227	.37	.43
Republican, Junction, Kans	23,100	840	7,887	484,963	.30	.35
Republican, Superior, Nebr	12,490	970	a3,080	a189,360	a.14	a.16
Blue, Manhattan, Kans	43,430	4,540	16,866	1,037,050	1.78	2.05
Kansas, Lecompton, Kans	81,400	11,450	38,291	2,354,422	.65	.75

METHODS FOR PREVENTING DAMAGE BY FUTURE FLOODS.

It must be remembered that the storm that caused the flood on Kansas River was not a local one of short duration, but extended over a considerable portion of six States and Territories and lasted nearly half a month. The storm was brought about by a rare combination of atmospheric pressure, temperature, moisture, and wind

velocity. Each of these was practically at or near its maximum influence at the time. There is no reason why this combination of rainfall factors may not occur again, and when it does occur a storm of approximately the same magnitude may be expected. Nothing whatever can be done to prevent its occurrence. It is sixty years since the last great flood, but there is no way of predicting when the next one will occur.

Although nothing can be done to prevent the occurrence of such a storm, much can be done to lessen the height of the resulting flood and the loss of property due to it. It is self-evident that to reduce the magnitude of a flood we must in some way either store the water that falls or hasten its flow from the drainage area, or both.

SURFACE AND GROUND STORAGE.

Nature has two ways of storing water, viz, on the surface of the ground and in the ground. In mountain regions water is stored for several months as snow and ice, and in the early summer passes down the streams. In regions of higher temperature surface water is stored in lakes, ponds, and swamps. Where streams are fed from mountain snow or lakes the distribution of flow throughout the year is very different from that of streams without surface storage. Surface storage in some sections of the country is vastly greater than that in others. In Maine, for example, there is 1 square mile of lake and pond area to every 14.3 square miles of land area. In other words, 7 per cent of the surface is natural water storage. On the prairies of the Kansas River watershed the natural surface storage is so small that it amounts practically to nothing.

EFFECT OF GROUND STORAGE ON FLOW OF STREAMS.

A rainfall of a depth of, say, 2 inches on a watershed disappears in three ways. A part flows over the surface and forms what is called surface run-off. A second part evaporates, thus passing off unseen from the surface of the ground and from pools of water. A third part sinks into the ground and is held for a greater or less time, but eventually is either absorbed by vegetation and passes off into the air through leaves or percolates through the ground into the streams. The total run-off—that which is left of the rainfall after the loss by evaporation and plant growth—varies from about 50 per cent in the eastern part of the country to almost nothing in the arid portions.

The amount of water that the soil will hold when saturated or full, depends on its porosity and depth. Let us suppose that the ground at a certain place is composed of a 6-foot layer of sand whose porosity is $33\frac{1}{3}$ per cent. Each foot in depth of this material will contain, when fully saturated, 4 inches of water, and the 6-foot layer will contain 2 feet depth of water. It will rarely be possible to store, even in sand, this amount of water, because it is seldom empty. It always holds back or contains some water.

Ordinary soils differ very much from sand in the amount of rainfall they will absorb and in their power of holding back water. Some soils are more porous than sand. Whitney[a] has shown from determinations in 23 localities, that the voids in the soil in Maryland and South Carolina are 48.73 per cent. The rate of percolation through soil is much slower than through sand. King has shown by experiments on columns of sand and soil, that the rate of percolation through sand of a degree of fineness such that it will pass a screen of 40 meshes to the inch and be retained on one 60 meshes to the inch, is 12½ inches per hour, when the water is kept 2 inches in depth on it. The rate through clay loam under the same conditions was only 0.07 inch per hour; and through black marsh soil, only 0.03 inch per hour.

The retaining power of a soil depends on the size of the grains, distance above standing water, and length of time since last rainfall. King found from his experiments on columns of sand 10 feet long that when the size of the grains was 0.0186 of an inch in diameter 3.77 per cent of the water was retained after 111 days. When the size of the grains was 0.0045 inch in diameter 7.57 per cent was retained after 111 days of percolation. The retaining power of true soils is much greater than that of sand.

The ground storage of the Kansas River basin is large as compared with that of other sections of the country. Consider, for example, the Blue River basin, which has the largest rainfall of any watershed in the Kansas basin. The mean annual rainfall is about 29 inches. The depth of run-off for the five years from 1897 to 1901, is about 2 inches. That is, only about 7 per cent of the rainfall on this basin appears as run-off, the remaining 27 inches were either evaporated directly or stored in the ground for a time and then passed off into the air through vegetation. A considerable part of the run-off was also stored in the ground for a time.

The surface run-off from sandy areas is very small, but from steep, bare, rocky areas it is rather large, in some cases 80 per cent or more of the rainfall. The water thus stored in the ground is given out gradually to the streams. It reduces their flood flow and increases their flow during periods of small rainfall. At the end of a long dry season the ground storage may be very low. A considerable amount of rain is necessary to make good this ground storage before the rain can increase the flow of the stream to any considerable extent. A rainfall of, say, 2 inches in depth falling moderately at the end of a two months' drought may raise a stream at a given place only a foot, whereas the same rain falling on that watershed when the ground storage is normal will raise the river at that place perhaps 8 or 10 feet. The magnitude of a flood, therefore, depends to a considerable extent on the amount of ground storage available at the beginning of the flood. If the storm in the Kansas River basin of May 16 to May 31

[a] Whitney, Milton, Some physical properties of soils in their relation to moisture and crop production: Bull. U. S. Weather Bureau No. 4, Dept. of Agric., 1892, pp. 23-28.

had come at the end of a long dry season the resulting flood would not have been so large and destructive.

What can be done to increase ground storage and reduce the amount of water flowing over the surface? Evidently, anything that will stop or retard the surface flow will accomplish this result. If the surface of a watershed were everywhere a plane of the same inclination the water would flow off it more rapidly than if the surface were very rough. Water collects in the depressions on the surface and remains there sufficiently long, in most cases, to be completely absorbed by the ground; hence the small ponds which hold water for a few days only are of value in increasing ground storage and reducing the magnitude of floods. Forests and small areas of trees also have an effect in holding back surface flow and increasing ground storage; also in transmitting moisture from the earth to the air through the leaves.

Cultivation of the soil increases the ground storage. Loose, mellow areas will absorb more rain than the same area covered with a tough sod or a bare uncultivated ground.

ARTIFICIAL STORAGE.

Next to natural lake storage artificial storage by dams constructed at a reasonable cost on the headwaters of streams is the most obvious and effective way of reducing floods and regulating the flow of streams. It is the more effective because it not only reduces the flood and the loss due to it, but it saves the water for irrigation, municipal supplies, and industrial purposes. Unfortunately there are some watersheds whose topography is such that large storage reservoirs can not be constructed at a reasonable cost. The Kansas River basin is one of this kind. A storage reservoir on the river or any of its principal tributaries would be very expensive to construct, would flood much valuable land, and would reduce the magnitude of the flood very little.

An enormous volume of water flowed down Kansas River during the recent flood. The estimated flow passing the Lecompton gaging station on May 31 is 225,400 cubic feet per second. The cross section area there, having a velocity of 4 or more feet per second, is estimated to be 28,000 square feet (width 1,000 feet, mean depth 28 feet). The mean velocity is, therefore, 8.05 feet per second or 5.5 miles per hour. The distance from Manhattan to Kansas City is about 125 miles; 22.7 hours, therefore, were required for the water to travel at this rate between those two places. If the whole Kansas River Valley between these places had been a reservoir 125 miles in length, it would have been filled in 23 hours on May 31, 1903.

The enormous volume of water passing down this river may be illustrated in another way. The flow of 225,400 cubic feet per second is equivalent to a volume of 455,348 acre-feet in twenty-four hours. The total drainage area of Kansas River at Lecompton is 58,550 square

miles. The ratio of 455,348 to 58,550 shows the average number of acre-feet of water flowing on May 31 from each square mile of the whole drainage of Kansas River. In other words, if there had been a storage reservoir 1 acre in area and 7.8 feet deep on every square mile of the whole Kansas River drainage basin the water flowing down Kansas River on May 31 would have filled every one of them. If on each half section of the whole of the Kansas River basin there had been a reservoir 1 acre in extent and 4 feet in depth the water flowing in Kansas River on May 31 would have been sufficient to nearly fill each one of them.

RAPID REMOVAL OF WATER.

Water that can not be held back for some days by storage should be passed out of the watershed as rapidly as possible if the danger of floods is to be reduced to a minimum. Anything that retards the flow backs up the water and causes a rise in the river and consequent overflow of area.

Dams and river bends have a decided retarding effect. The water has a longer distance to travel by reason of the bends and thus a longer time is necessary for it to pass out of the watershed, and the velocity of the water is reduced by striking the bank at each bend.

Bridges across a stream are sometimes a very fruitful cause of increasing high water. They restrict both the width and depth of the waterway. The partial or total destruction of nearly all the bridges across Kansas River is very positive evidence of this fact. The Engineering News of September 17, 1903, contains the following in regard to the restriction in width of Kansas River at Kansas City:

> The Kaw [Kansas] River near the mouth has been gradually encroached upon for years, the abutting property owners filling in and reclaiming the land for industrial purposes. The normal width of the river from bank to bank is about 800 feet. At the James street bridge, just above the mouth, this channel has been narrowed down to about 450 feet between abutments. The effect of this construction is also shown by the fact that the high-water mark at the Turkey Creek pumping station, about 2 miles above the mouth of the river, was 10 feet above that at the mouth. If the river had been given its natural width through the city, the water would not have risen so high by from 6 to 8 feet in the upper districts as it did under the existing conditions. It is evident that the clear channel of the Kaw should be made from 800 to 1,000 feet wide from Argentine to the Missouri River.

A channel 200 feet wide was cut around the north end of the bridge at Lawrence by the flood. This channel is being closed by an embankment. The closing of this opening by a solid structure, thus reducing the waterway to its old dimensions, which the flood has shown to be too small, is ignoring one of the obvious lessons of the flood. It would have been better to have built a structure here with gates for holding back the water for power purposes, which could have been opened in time of flood to give a larger waterway.

The masonry arch bridge across the Kansas at Topeka is a good example of the evil resulting from restricting too much the waterway. The flood passed several feet in depth over both abutments of this bridge, washing out one of them and badly damaging the bridge and the other abutment. The old bridge at this place had six spans of 150 feet each; the present masonry structure has reduced the waterway nearly one-half, and has had much to do with the overflow at Topeka. There are several new bridges across the Kansas that restrict its flow much more than the old ones. The small saving in the cost of a bridge by reducing its length 100 feet is paid many times over in the loss of property caused by the overflow due to restricting the waterway. A very obvious means of reducing the height of future floods is to widen the channel in the cities through which the river passes. The minimum width and the number and size of bridge piers should be prescribed by law.

This flood has shown that many of the bridges across Kansas River before the flood were too low. The new ones should be higher than the old ones or of a design such that the bridge can be raised or swung out of the way during very high water and leave a clear channel for drift to pass.

CUT-OFFS AND OVERFLOW CHANNELS.

The effect of a cut-off is to shorten the distance traveled by the water, to reduce the loss of head due to friction, to increase the velocity of flow, and thus to reduce the height of the flood. If the bed and banks are composed of firm material that will not be worn away by the current, this plan works well, but for rivers flowing through alluvial material, such as Mississippi, lower Missouri, and Kansas rivers, this remedy is only temporary. The increased velocity will cause additional scours and the formation of new bends. Thus this process will continue until the river is restored to its former length. The studies of Humphrey and Abbott, and other engineers, of the Mississippi prove conclusively that this plan is not a good one for rivers flowing through alluvial bottoms.

Overflow channels increase the high-water waterway of the river and thus reduce the height of flood. They are equivalent to widening the main channel. Increasing the area of the main channel for a given amount, say 1,000 square feet, is, as a rule, more effective than one or more overflow channels of the same total area.

LEVEES.

Levees are proposed as a remedy for floods. This method is used on the Mississippi for securing the land against inundation. It was begun in 1717 at New Orleans by riparian owners and carried on until 1850 by the States through which the river passes. Since this date levee building has been carried on by cooperation of the State and National governments. All of the engineers who have made a

study of methods of preventing floods in the Mississippi basin recommend this as the only feasible method of accomplishing the desired result. The levees or earth banks on each side of the river approximately parallel to the course of the stream, limit the width of the stream, increase somewhat the velocity and consequent scour, and the height of high water.

Three objections are raised to the use of levees: First. That they can not be made strong enough. Second. The cost is too great. Third. They raise the bed of the river by depositing in it the silt that otherwise would be deposited on the flooded land.

It is difficult to see how the third objection can be true. The increased velocity will cause a scour rather than a deposit of material on the bed. Furthermore, a careful study of the effect of the levees along Mississippi River fails to show any appreciable rise in the bed of that river.

The numerous and frequent breaks in the levees during the floods apparently show that there may be some truth in the first objection. This is only apparent, however, as earth embankments can be made so strong and durable that they will not fail and so high that the greatest floods can not pass over them. Many earth dams and embankments have been built much higher than any of the levees and have stood the test of many years. The strength and durability of such a structure is only a matter of cost.

Whether or not the levee method of preventing inundation of the Kansas River bottom is feasible can be definitely decided when the cost and benefits to be derived are known. Ordinary floods cause overflow of some of the bottom lands along the river. The flood of 1903 was 17 feet higher than ordinary floods, so that the levee will need to be 20 feet high in some places. The cost per mile in length of an embankment of this height, with a top width of 5 feet and side slopes of 2 horizontal and 1 vertical, will be as follows, assuming the cost to be 20 cents per cubic yard:

$$\frac{900 \times 5280}{27} \times .20 = \$35,200.$$

If the embankment is only 10 feet high, 3 feet top width, side slopes of 2 horizontal to 1 vertical and the cost per cubic yard 20 cents, the total cost per mile of the embankment will be $8,993.

The benefits to be derived will scarcely justify either of these expenditures except at or near cities.

SUMMARY.

The highest recorded flood at St. Louis occurred in 1785. The flood of 1844 came within 1 foot of the height reached in 1785, and the flood of 1903 came within about 3 feet of the stage of 1785.

The highest flood on record at Kansas City was in 1844. The flood

of 1903, although it did not reach that height, was vastly more destructive on account of the country being more thickly settled and improved.

The Kansas River gage records at Lawrence and Lecompton cover a period of twenty-two years. The flood of May and June, 1903, was 17½ feet higher than at any other time within this period. The volume flowing in Kansas River on May 31 was nearly three and a half times larger than that at any time during these twenty-two years.

The period of excessive rainfall extended from May 16 to May 31. Water fell to a depth of 10 inches in some places during this time. The rainfall at some of the stations for the whole month of May was 17 inches, so that the precipitation for the first half was above the normal, the ground was well supplied with moisture, and the flow of the streams was above the normal. The mean annual rainfall of the Kansas River basin varies from about 35 inches in the eastern part to 12 inches in the western part. About 14 per cent of this mean annual rain falls in May, but in May, 1903, nearly 50 per cent of the mean annual rainfall occurred.

The drainage area of Kansas River is a long narrow strip between the Platte and the Arkansas River drainage areas. It is rolling prairie without mountains, with broad, shallow valleys, fertile soil, and very little forest. There is very little service storage, but the ground will absorb a large quantity of water. The flood flow of the stream is large and the minimum flow small, as compared with streams in the eastern half of the United States.

The Kansas River bottom is mainly sandy, alluvial material from 1 to 4 miles in width; the banks as a rule are not more than 10 feet in height and are easily eroded by the current.

For eleven days, May 28 to June 7, inclusive, the water was from 1 to 12 feet higher at Lecompton than it had been before in twenty-two years; and for twenty days, May 23 to June 11, it was from half a foot to 17½ feet higher than ordinary high water. The whole river bottom was flooded, crops were destroyed, stock drowned and swept away; railroads were under water, partly washed away and undermined; wooden buildings were lifted from their foundations and swept down the stream; brick and stone buildings crumbled and fell; bridges were swept away and cities left without drinking water, light, or power; traffic was almost entirely suspended for two weeks. The property loss in Kansas and Kansas City is estimated to have been $22,000,000.

Nothing can be done to prevent the recurrence of a storm like the one from May 16 to 31, 1903, which caused this enormous loss, and no person can predict the time of occurrence of the next one.

To reduce the magnitude of the resulting flood, the rainfall must quickly be passed out of the watershed or stored in it. The opportunities for surface storage in the Kansas River basin are small, for the

valleys and depressions are broad and shallow, the amount that can be stored small, and the cost of storage large. On the headwaters, where the possibility of storage is greatest, the rainfall is smallest. By a glance at the map it will be seen that nearly all of the rain that caused this flood fell on the lower half of the watershed, where the possibilities for surface storage are the least.

If there had been a reservoir 1 acre in extent and 4 feet in depth on each half section of land in the whole Kansas River basin, the volume of water flowing down Kansas River on May 31 was sufficient to have nearly filled all of them.

Ground storage in the Kansas River basin is large; it can be increased by cultivation of the soil and growth of forests. Not more than 7 per cent of the rainfall on this drainage area appears as run-off in ordinary years; the remaining 93 per cent disappears as evaporation directly from the surface and indirectly through plant growth.

Any obstruction in the form of a dam, bend in the channel, or reduction of the normal width retards the flow and increases the magnitude of a flood. The partial destruction of nearly all the bridges across Kansas River is positive evidence that they restricted the flow too much. The normal width of Kansas River at Kansas City has been reduced nearly one-half, and as a result the recent flood was about 10 feet lower at the mouth than about 2 miles up from the mouth.

Reducing the length of the channel by straightening it will reduce the magnitude of the flood, but in alluvial bottom land the increased velocity causes scour and the formation of new beds, so the remedy is only temporary.

Overflow channels give an additional waterway and reduce the height of floods, but are seldom practicable.

Levees have proved to be the best means of preventing inundation in the Mississippi River basin; they do not reduce the height of floods, but simply prevent overflow. The cost of levees sufficiently high and durable to prevent overflow of the Kansas River bottom during a flood of the magnitude of that of 1903 would be too expensive to build except in and near cities.

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